pumping at a higher rate than normal. It is believed that the rewinding may actually have resulted in a higher rpm causing this pump to operate at a higher level.

These later storms produced overflows of such volumes that the overflow valve needed to be opened as much as sixteen turns. The pumps were noted to operate far up on their curves (4,000 gpm+ each) without any detrimental effects due to suction head loss, cavitation, etc. Thus it is concluded the existing pumps may be able to be utilized for greater flows simply by decreasing discharge head loss.

Erie City / Millcreek Township Sewer Authority Connection / Manor Drive

Discharge volumes from the Kearsarge pump station enter the Pittsburgh Avenue sewer and ultimately enter the City of Erie for transport to the Erie Wastewater Treatment Plant at the Manor Drive interconnection. The Manor Drive interconnection consists of two separate interceptor sewers and thus two meters. One is entitled "Manor Drive" and the other is entitled "The Boyer meter." The total of the flow at both meters makes up the total flow added at that connection.

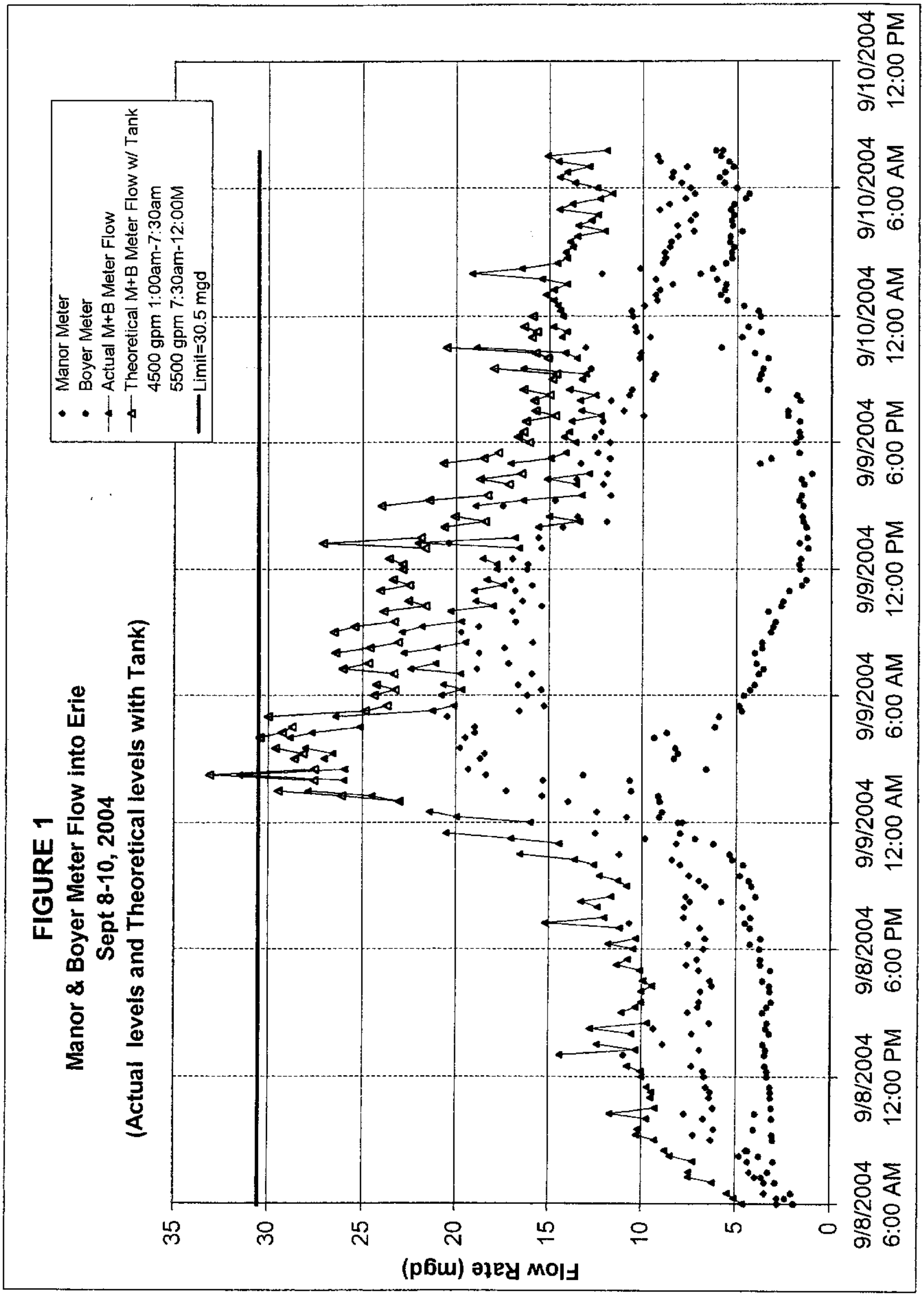
Flows from Summit and Fairview make up part of that flow in addition to the Millcreek Township flows. Millcreek flows are determined by subtracting the Summit and Fairview flows from the total volumes. Under the City agreement Millcreek is permitted a total of 23.27 MGD as their total discharge rate. Summit is permitted 3.9 MGD and Fairview is permitted 3.33 MGD for a total of 30.5 MGD. Compliance with the agreement is based upon the rate, not the total. Raw data regarding peak instantaneous flows at both metering stations were obtained from Erie City personnel immediately following the September 9, 2004 storm. Raw data consists of the depth of flow and the velocity of flow through the two 36-inch meters. Observation times were not obtained from City personnel at that time but since then (November), we have received the needed data on 20-minute intervals. The information is graphed on Figure 1. The flow rates at each of the two stations is also given on the figure. The flow rate at 2:40 a.m. (not shown) is the only time when the total exceeded the allowable. That value is believed to coincide with a flow rate of 5,300 gpm at 1:30 AM from Kearsarge (800 gpm higher than design) caused by a wet well surcharge.

An earlier power failure had allowed the wet well to fill to 20 ft. above the normal level. This additional suction head pushed the pumps' operation forward on their curve resulting in the unusually high discharge rate. If flows were maintained at design level, the discharge rate into the City would have been below the agreement maximum.

Millcreek's limit of 23.27 MGD (76.3% of the total) was exceeded for about 1 hour and fifteen minutes and would have been exceeded for 3 hours at design flows of 4,500 gpm.

Bypass Plug Dislodgement

The discovery of the plug dislodgement is discussed under the December 31st storm.



/MTSA/Manor_Boyer/Manor_Boyer_Oflow Sept2004.xls

RESULTS

Overflow Events

July 31, 2004 Overflow Event

This event's observations were revisited following the December 31st event to determine if there was any sign of an impact on the station which might be attributed to an influx of stream waters. None were evident and a comparison of upstream meters with pump station flows confirms it. The flows pumped from the station along with the bypass volumes during the July 31, 2004 incident are found on Table 1. Flows pumped were calculated to vary from a high of 7,155 gpm to a low of 4,050 gpm. This will dictate the total amount of pump capacity that will be necessary to handle a similar storm in the future without bypass. In this event the amount of flow bypassed and the amount of flow pumped forward are also given in Table 1 in the fifth and sixth columns. In the future all flows will either have to be pumped forward or will have to be pumped to storage. If, as intended, 4,500 gpm is to be pumped forward, as much as 2,655 gpm would have needed to have been pumped to storage. This compares to the 2,340 gpm rate proposed to be provided as the storage pumping capacity in the original report.

The flows bypassed were those values that were not pumped forward during this process. Those flow volumes will exceed the required amount for storage because the forward flow, when the bypass is open, drops below the future 4,500 gpm value. Forward flow values were as low as 1,900 gpm during the bypass event. The total flows bypassed at the station equal 2,253,000. An additional 42,000 gallons of wastewater was bypassed utilizing portable pumps to relieve overloading in the system. The need for these pumps will also be abated by the project by increasing capacity of sewers. Thus, that 42,000 gallons will ultimately reach the pump station. Total bypass volumes equaled 2,295,000 during this storm event.

Table 2 gives the required storage to handle a similar storm in the future with 4,500 gpm forward pumping rate. This table shows the actual flows from the station with assumed pumpage rate forward and calculates the amount of storage required for each flow increment observed at the station. It can be seen in column 3 that the station pumping rates varied throughout the event. This then would vary the storage requirement on an hourly or some other time interval. The eighth column gives the storage requirements as a summary. The reader will note that under the first event beginning at 5:30 a.m. approximately 556,000 gallons of storage would be required. However, between rain events the storage would have been unable to empty and would have still contained 430,000 gallons when the second storm began. The second event added an additional 845,000 gallons. The total in the tank would have reached 1.275 MG. If this figure is adjusted for the flows bypassed utilizing portable pumps in the system, calculated at approximately 40,000 gallons, total storage requirements would equal 1,315,000 gallons. Adjusting further for growth until 2014, an additional 10,000 gallons per hour average flow will be seen at the station. The required capacity in the storage increases to 1.525 MG over the period of the overflow event.

TABLE 1

KEARSARGE PUMP STATION
JULY 31 - AUGUST 1, 2004
BYPASS VOLUME

Date	Time	Flow 1.35 x meter	Pressure	Forward Flow	Bypass	Bypass x 10^{-3}
7/31	5:30 a.m.	4,725				
	6:30 a.m.	7,155	37	3,150	4,000	120
	7:00 a.m.	6,615	43	3,825	2,800	84
	7:30 a.m.	6,750	39	3,825	2,950	88
	8:30 a.m.	5,400	30	1,912	3,500	105
	9:00 a.m.	4,725	47	3,825	900	27
	9:30 a.m.	5,800	43	3,825	2,000	60
	10:00 a.m.	5,800			2,000	60
	10:30 a.m.	5,670	39	3,710	1,950	60
	11:00 a.m.	5,130	44	3,800	1,350	40
	11:30 a.m.	4,860			1,000	30
	12:00 p.m.	4,995			1,200	36
	12:30 p.m.	5,130			1,350	40
	1:00 p.m.	4,860	41	3,800	1,000	30
	1:30 p.m.	4,320	45	3,800	500	15
	2:00 p.m.	4,050			0	
	6:00 p.m.	4,050			0	
	6:30 p.m.	4,725			950	30
	7:00 p.m.	5,400	38	3,200	2,200	66
8/1	12:00 a.m.	7,000			3,800	1,140
	1:00 a.m.	5,400			2,200	132
	2:00 a.m.	4,725			1,500	90
	2:30 a.m.	4,050			0	
	3:00 a.m.	4,050			0	
				TOTAL		2,253,000
				for pumps		42,000
						2,295,000

TABLE 2

KEARSARGE PUMP STATION
JULY 31 - AUGUST 1, 2004
NEEDED STORAGE (@4,500 gpm)

Date	Time	Flow 1.35 x Meter	Assumed Pump Rate (gpm)	Transfer	Storage Δ Time (hr.)	Storage Δ Storage x 10^{-3}	Storage ∑ Storage
7/31	5:30 a.m.	4,725	4,500				
	6:30 a.m.	7,155	4,500	2,655	1.0	159	159
	7:00 a.m.	6,615	4,500		0.5	80	239
	7:30 a.m.	6,750	4,500		0.5	68	
	8:30 a.m.	5,400	4,500		1.0	54	
	9:00 a.m.	4,725	4,500		0.5	7	
	9:30 a.m.	5,800	4,500		0.5	39	
	10:00 a.m.	5,800	4,500		0.5	39	
	10:30 a.m.	5,670	4,500		0.5	35	
	11:00 a.m.	5,130	4,500		0.5	19	
	11:30 a.m.	4,860	4,500		0.5	11	
	12:00 p.m.	4,995	4,500		0.5	15	
	12:30 p.m.	5,130	4,500		0.5	19	
	1:00 p.m.	4,860	4,500		0.5	11	556
	1:30 p.m.	4,320	4,500		0.5	-5	
	2:00 p.m.	4,050	4,500		0.5	-13	
	6:00 p.m.	4,050	4,500		4.0	-108	430
	6:30 p.m.	4,725	4,500		0.5	7	
	7:00 p.m.	5,400	4,500		0.5	27	
8/1	12:00 a.m.	7,000	4,500		5.0	750	
	1:00 a.m.	5,400	4,500		1.0	54	
	2:00 a.m.	4,725	4,500		0.5	7	1,275
	2:30 a.m.	4,050	4,500		0.5	-13	
	3:00 a.m.	4,050	4,500		0.5	-14	1,250

Adjusting for 2014 add 10,000/hr. = 210,000 gallons
Adjusting for manhole pumps 450 x 60 x 1.5 = 40,000
1,275 + 250 = 1.525 MG

To summarize, the July 31, 2004 overflow event would require a 1.525 MG storage tank, 4,500 gpm forward pumping rate, and a 2,655 gpm storage pumping rate in order to manage a similar overflow event in 2014.

September 9, 2004 Overflow Event

This event was also revisited to determine if it was impacted by the cross connection found later. It was concluded that it had been. Unfortunately upstream meters were damaged in the flooding but the overflows characteristics demonstrate that there was undoubtedly an influence due to an influx of stream water. That is best explained by the cross connection found in December. The impact on the original observations is discussed at the end of this section.

This event was a response to a long-term storm event which was a residual to a hurricane. The assumptions and calculations included in this summary were obscured by several incidents during the storm event.

The first impact was the initial power failure which caused recovering pump rates to exceed future forward flow rates due to the extremely high suction head caused when the wet well filled to within 2 ft. of the top.

The greatest impact was caused when a tree was felled just downstream from the plant taking with it an old abandoned bridge abutment. This blocked the flow in Walnut Creek causing a backup of water above the previously defined 500-year flood elevation. The backup actually ponded water up to the pump station gates. There were several manhole lids that had been left ajar to allow sewage flows to be transferred from one to the other during a construction project. As a result these flood waters were capable of directly entering the Authority interceptors just upstream of the pump station. This incident was documented between 4:30 a.m. and 7:30 a.m. before stream flows receded sufficiently to lower water levels upstream of the blockage below the stream's banks. The pump station flows increased rapidly at about 4:30 but did not decrease proportionally until 10:30 a.m. The assumption is that this flow increase that occurred during this period was caused by these flood waters. This assumption is further supported by the fact that the flows during the period exceeded the capacity of the incoming sewers upstream of the station under a full surge condition. Thus flows had to be coming from a point near the pump station. The following calculations assumed that the flow increment increase noted in the time frame above was not applicable for use in calculating what would be needed as storage. It was used in calculating the overflow volumes but was not used in calculating the storage or the pumping needed. It is an example, however, of the need to provide contingencies such as increased forward flow and backflow preventer to address such anticipated events at the station since gravity overflow without basement flooding is not a possibility.

The third incident impacting this overflow event occurred towards the end of the storm and was caused when a pump motor failed. This pump was then out of service for a period of time and before reaction could take place to increase the station pumping

capacity, the wet well backed up again. It is not certain that the storage needs after that event are properly characterized. It is believed that they may be too high.

The September 9th storm event began at approximately 5:00 p.m. on September 8th. One inch of rain had fallen by 11:00 p.m. on the 8th and an additional 2-inches of rain had fallen by 2:00 a.m. on the 9th. The total amount of rain that fell within a twelve-hour period incorporating the subsequent overflow are given in the attached summary entitled "Rainfall Data" as between 4 and 5-inches. This is equivalent to a rainfall having a return frequency of between 30 and 70 years. We have assumed this to be a 50-year return frequency storm. The entire event is summarized in Appendix B entitled "History" beginning at 5:00 p.m. on September 8th and ending at 3:00 a.m. on September 10, 2004. The overflow volumes noted during the event are found in Table 3. The overflow began at 2:00 a.m. and ended at approximately 12:15 a.m. on the 10th. The overflow event was extended because of pump #1 failure at 3:30 p.m. and was exasperated by the damming of the stream and the partially open manholes. The volume bypassed totaled 6.02 MG with 813,000 gallons being created by portable pumps pumping from manholes and 600,000 gallons originally assumed to have been caused by the stream backup.

The bypass was extended from 4:30 p.m. on the 9th until midnight because of the pump failure during which time there was a total of 556,000 gallons discharged. Only 48,000 gallons of that would have needed to be stored at 4,500 gpm of forward pumping.

The pump needs for storage equal the observed flows from the station at times other than during stream backup plus the calculated portable pump output located at the various manholes. That figure is found in Table 4 and equaled 7,200 gpm plus 1,400 or 8,600 gpm. Subtracting 4,500 gpm, the needed peak storage pumping rate equals 4,100 gpm. Adding flows to that anticipated by 2014, that number becomes 4,540 gpm.

Although the storage requirements, calculated following the storm, are heavily impacted by later observations of stream water influx, they are summarized here to allow the reader to assess our assumptions beginning in the next paragraph. The original storage requirement calculations are summarized in Table 4 and totaled 2,772,000 gallons over the entire period which includes 813,000 total gallons calculated to have been discharged from the three pumps distributed throughout the system to pump from manholes. If 2014 flows are added, an additional 200,000 gallons or 2.97 MG of storage would have been necessary.

The above observations made during and immediately after this storm were impacted by discoveries during the December 31, 2004 storm when it was discovered that an upstream overflow plug had been dislodged creating a cross connection between the sanitary sewer and the stream. That cross connection is located at approximate elevation 310 which is approximately 20 ft. above the floor of the wet well at the third landing. Its impact on the observations are first that the influx of stream flows into the station are now believed to have been through the displaced plug in the overflow and the duration and volume of its influence is believed greater and longer than originally assumed since the stream level

TABLE 3

KEARSARGE PUMP STATION
SEPTEMBER 9, 2004 BYPASS FLOWS

Date	Time	Metered Flows (GPM)	Corrected Flows (GPM)	Discharge Pressure (psi)	Forward Flows (GPM)	Bypass Flow GPM	Bypass Flow Δt	Bypass Volume gallon x 10^{-3} PS	Bypass Volume gallon x 10^{-3} Manhole
9/9/2004	1:30 a.m.	4,000	5,300				0		
	2:30 a.m.	5,400	7,200	40	3,400	3,800	1 hr.		
	3:30 a.m.	5,400	7,200		3,400		1 hr.	456	36
	4:30 a.m.	6,700	8,900	32	2,000	6,900	1 hr.	414	48
	7:30 a.m.	6,500	8,800		2,000	6,800	3 hr.		
	9:30 a.m.	6,400	8,800		2,000	6,800	2 hr.	2,040	
	10:30 a.m.	6,200	8,300		2,000	6,300	1 hr.	378	504
	11:00 a.m.	4,500	6,000		2,000	4,000	1/2 hr.	120	
	3:00 p.m.	4,500	6,000		2,000	4,000	4 hr.	960	
	4:00 p.m.	4,000	5,400		2,000	3,400	1 hr.	204	
	4:30 p.m.	3,500	4,700		2,000	2,700	1/2 hr.	81	
	6:00 p.m.	3,500	4,700	(36) +	2,800	1,900	1-1/2 hr.		225
	8:00 p.m.	3,500	4,700			1,900	2 hr.	396	
	9:00 p.m.	3,000	4,050	(32) +	3,400	650	1 hr.		
	12:00 a.m.	3,000	4,050		3,400	650	3 hr.	156	
	12:10 a.m.				0				
TOTAL								5,208	813

\+ Estimated based on past history with overflows (i.e.) 13 turns

Manhole (MH) Bypass	Rate (gpm)	Hours Full-time	Hours Part-time	Gallon Total
52nd & Zimmerly	600	8	7.5	378,000
Patton & Church	200	7	7.5	129,000
Larchmont	600	6	7.5	306,000

TABLE 4

KEARSARGE PUMP STATION
SEPTEMBER 9, 2004 STORAGE FLOWS NEEDS

Date	Time	Metered Flows (GPM)	Corrected Flows (GPM)	Corrected Flows w/o Flood Waters	Forward Flows (GPM)	* Storage w/o Flood Pump Flow		Δ Storage Volume * (x 1000)		Σ Storage Volume (x 1000)
							Δt	PS	Manhole	
9/9/2004	1:30 a.m.	4,000	5,300	5,300	4,500	800	.5 hr.	24		24
	2:30 a.m.	5,400	7,200	7,200	4,500	2,700	1 hr.	162		186
	3:30 a.m.	5,400	7,200	7,200	4,500	2,700	1 hr.	162	36	384
	4:30 a.m.	6,700	8,900	7,200	4,500	2,700	1 hr.	162	48	594
	7:30 a.m.	6,500	8,800	7,200	4,500	2,700	3 hr.	486	252	1,332
	9:30 a.m.	6,400	8,800	7,200	4,500	2,700	2 hr.	324	168	1,824
	10:30 a.m.	6,200	8,300	6,700	4,500	2,200	1 hr.	132	84	2,040
	11:00 a.m.	4,500	6,000	6,000	4,500	1,500	1/2 hr.	45	15	2,100
	3:00 p.m.	4,500	6,000	6,000	4,500	1,500	4 hr.	360	120	2,548
	4:00 p.m.	4,000	5,400	5,400	4,500	900	1 hr.	54	30	2,664
	4:30 p.m.	3,500	4,700	4,700	4,500	200	1/2 hr.	6	15	2,685
	6:00 p.m.	3,500	4,700	4,700	4,500	200	1-1/2 hr.	18	45	2,748
	8:00 p.m.	3,500	4,700	4,700	4,500	200	2 hr.	24		2,772
	9:00 p.m.	3,000	4,050	4,050	4,500	0	1 hr.			
	12:00 a.m.	3,000	4,050	4,050		0	3 hr.			
	12:10 a.m.									
TOTAL								1,959	813	2772 **

* Assumes flow increment beginning at 4:30 a.m. equaling 1,700 gpm was caused by partially open manholes flooded by stream backup caused by felled tree and bridge abutment. That volume is not included in the storage calculations.

** 2004 storage volume = 2,772,000; 2014 volume = 2,957,000 gallons

simply dropping to within its banks would not have been sufficient to abate the influx through the abandoned overflow. At 10:00 AM on September 10th, the stream level was just below the top of the old wing wall which would still cause an influx (Appendix C) thus we estimate the stream continued its influence through noon. The stream influence is now estimated at 1,263,000 gallons. Table 5 gives the revised calculations. Estimated storage volumes needed to have addressed this 40 to 50-year storm are reduced to 2,142,000 gallons in 2004 and 2,302,000 in 2014.

September 17, 2004 Overflow Event

This event was also reviewed in light of the December 31, 2004 findings. Again there was no upstream meter information. The discussion of the assumed impact of a stream cross connection follows the original discussion of the storm.

This event was also due to the residuals of a hurricane. It began at approximately 5:10 a.m. and lasted to 7:00 p.m. on the 17th. Three rainfall gauges were consulted. It appears the rainfall was approximately 1.6-inches total during that time frame. Approximately ½-inch had fallen by 7:30 a.m., another inch by 4:00 p.m. with another 1/8-inch by 7:00 p.m. The total rainfall was not extensive nor did it approach any rainfall frequency occurrence greater than one-year. The one rainfall gauge in Summit Township which is the only gauge in the Kearsarge drainage basin implies significantly more rain, 3-inches over the same time frame with approximately 1.8-inches falling in the period 5:10 a.m. until 11:00 a.m.

This overflow event was complicated by several power failures attributed to the breaker at the station. The first one began at 3:50 p.m. and was finally resolved by 4:10 p.m. There were similar events at 7:00 and another at 7:15 but response time was greatly improved.

Flows peaked at 7,425 gpm at 10:30 and had dropped to 5,400 gpm at approximately 4:00 p.m. at the time of the power failure. Higher values were recorded when the power was returned to pump due to the built up residual flows, but they returned to 5,400 gpm. However, at 5:00 p.m. they increased to 7,000 gpm and remained at that rate for two hours until they began to steadily decrease until the bypass could be turned off at 11:30 p.m. Some of that increase is credited to the mall (being a Friday night) preparing for the evening's business between 5:00 and 7:00. Another explanation could be the rainfall shown by the Summit rain gauge but not by others. The Summit flow data, which is the only system data available to us at this time, showed a similar variation (although with a lesser amplitude) with a decrease in flows from 10:00 a.m. until 2:30 p.m. when flows began to increase showing two peaks, one at 4:00 p.m. and another at 7:00 p.m. Steadily decreasing values were observed for the rest of the evening. Their rain gauge showed rainfall of 0.35-inches from 5:00 p.m. to 7:00 p.m.

We again checked flows given by the discharge meter using the correction factor of 135% with the pump curves and the pressure recordings obtained. The pump curves match with the adjusted flows and observed pressures. Thus we are concluding that the

TABLE 5

KEARSARGE PUMP STATION
OVERFLOW
DATE: 9/9/04

ADJUSTING FOR STREAM INFLUX
Forward Flow 4,500 GPM (12 Midnight - 5:00 p.m.)

Time	Q Meter (gpm)	1.35 x Meter (gpm)	Interval (hours)	Forward Flow (gpm)	Stream Influx (gpm)	Manhole Pumps Bypass (gal)	Storage @ 4,500 (gallon)	Σ Storage (gallon)
12:00 M	3,200	4,320		4,500			0	
12:30 AM	3,800	5,130	(0.50)	4,500			630	18,900
1:00 AM	4,000	5,400	(0.50)	4,500			900	67,500
2:15 AM	4,000	5,400	(1.25)	4,500			900	
2:15 AM	5,300	7,155		4,500			2,655	119,475
3:00 AM	5,300	7,155	(0.75)	4,500		600	3,255	
3:30 AM	5,300	7,155	(1.00)	4,500	2,000			139,200
4:00 AM	5,200	7,020		4,500	2,000	800	1,320	
4:15 AM	6,700	9,045	(0.25)	4,500	2,000	1,400	3,945	59,175
4:30 AM	5,900	7,965	(0.25)	4,500	2,000	1,400	2,865	42,975
5:00 AM	6,500	8,775	(0.50)	4,500	2,000	1,400	3,675	110,250
6:00 AM	6,500	8,775	(1.00)	4,500	2,000	1,400	3,675	220,500
7:00 AM	6,500	8,775	(1.00)	4,500	2,000	1,400	3,675	220,500
8:00 AM	6,700	9,045	(1.00)	4,500	2,000	1,400	3,945	220,500
9:00 AM	6,400	8,640	(1.00)	4,500	2,000	1,400	3,540	230,700
10:00 AM	6,300	8,505	(1.00)	4,500	2,000	1,400	3,405	212,400
10:30 AM	6,000	8,100	(0.50)	4,500	2,000	1,400	3,000	90,000
11:00 AM	4,500	6,075	(0.50)	4,500	2,000	1,200	800	24,000
12:00 N	4,500	6,075	(1.00)	4,500	1,660	1,000	1,000	60,000
1:00 PM	4,500	6,075	(1.00)	4,500	1,200	800	1,200	72,000
2:00 PM	4,500	6,075	(1.00)	4,500	1,000	600	1,200	72,000
3:00 PM	4,500	6,075	(1.00)	4,500	950	400	1,000	60,000
4:00 PM	4,000	5,400	(1.00)	4,500	900	200	200	12,000
5:00 PM	3,500	4,725		4,500	850	0	0	
6:00 PM	3,500	4,725		4,500	800	0	0	
12:00 M					500			
12:00 N					0			
								2,052,075
For 2014 flows add 10,000 gpd/hr. = @16 hours = (160,000 gallon)								160,000
								2,212,075

kearsargepumpstationoverflow9-9-04Table 5.xl

flows given are accurate. On September 17th there was no distribution of portable pumps. The peak flow rates were 7,500 gpm at the station. Adding 440 gpm for 2014 design flows equals 7,865 gpm. Deducting 4,500 gpm from that equals approximately 3,000 gpm for diversion to a future storage tank versus the study recommendations of 2,340 gpm. Storage flows are also given in Table 6 calculated for the period of the storm. Assuming 4,500 gpm forward flow, storage requirements were originally assumed to equal 974,000 gallons for the September 17th overflow event.

When reevaluating the September 17th flows in light of the December 31, 2004 findings, some of the increase in flows beginning at 3:30 PM were attributed to the upstream sewer cross connection with stream waters. The first 700 gpm increase from 3:30 PM until 5:00 PM is believed due to stream surcharge. Approximately 1,000 gpm of the increase at 5:00 PM is credited to mall usage leaving an additional 600 gpm credited to the cross connection until 8:00 PM when flows begin to drop reaching zero from the cross connection by 11:00 PM. Cross connection flows then total 363,000 gallons (700 x 60 x 1) + 1,300 x 60 x 3 + 650 x 60 x 3). Needed storage adjusting for assumed storm water influx equals 611,000 gallons instead of 974,000 gallons.

December 23 and 31, 2004 Overflow Events

Neither of the two storms' bypass flows or storage needs approach the September 9th storm event. Their flows and calculated bypass volumes and storage needs are given in Tables 7 and 8. No corrections have been made in the tables for the stream influx impact in this storm since no stream observations were made until the end of the December 31, 2004 storm event, when the stream was noted to be near the top of the wing wall. Stream inflow was calculated at the end of the event, from visual estimates of flow depth and velocity, at 1,000 gpm. Attempts to correlate this data with an orifice formula using the open area of the gate (0-inches at the crown and 4-inches at the invert) were to no avail. Thus estimates of the inflow volumes from observed conditions were the only means available. Appendix D includes the calculations used.

Had an estimate been made of the stream influx, the change in storage required would have equaled 72,000 gallons on December 23rd and 420,000 gallons on December 31st. The stream influx impact is believed to have begun at 12:45 p.m. on December 23rd and at 10:30 a.m. on December 31st and is assumed to equal 1,000 gpm in both cases.

The depth of influx flow was estimated by the width of the flow in the 20-inch sewer invert and the velocity was estimated from the horizontal distance the plume traveled in falling the 9.6 ft. to the influent sewer below. The depth of flow was estimated at 4-inches and was used to determine the area of flow from tables. The horizontal distance the plume traveled in falling 10 ft. equaled 2 ft. (one-third of a second) which gives a velocity of 6 ft. per second. This equates to 1.28 MGD or approximately 1,000 gpm at 1.5 foot of head.

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TABLE 6

KEARSARGE PUMP STATION
SEPTEMBER 17, 2004 OVERFLOW
STORAGE/BYPASS/PUMPING

Time	Meter	1.35 x Meter	Interval	Storage * gallon x 10^{-3} Δ	Storage * gallon x 10^{-3} ∑	Pressure	Forward Flow	Bypass gallon x 10^{-3} Δ	Bypass gallon x 10^{-3} ∑
10:00 a.m.	3,700	5,000							
10:25 a.m.	3,800	5,130	0.5	19		40	3,700	43	
	5,500	7,425							
11:00 a.m.	4,800	6,480	0.5	72		41	3,700	99	
12:00 p.m.	4,500	6,075	1.0	94		38	3,600	148	
1:00 p.m.	4,000	5,400	1.0	54		38	3,600	108	
2:00 p.m.	3,500	4,725	1.0	14					
3:00 p.m.	3,500	4,725	1.0	14					
3.30 p.m.	3,500	4,725	0.5	14		43	3,700	168	
	4,000	5,400	0.5	27					
4:00 p.m.	4,000	5,400				30	2,000	102	
+ 5:00 p.m.	5,200	7,020	1.0	-		34	2,600	265	
6:00 p.m.	5,200	7,020	1.0	-		30	2,000	301	
7:00 p.m.	5,200	7,020	1.0	453		32	2,300		
8:00 p.m.	5,000	6,750	1.0	135		27	1,600	309	
8:30 p.m.	4,500	6,075	0.5	47		33	2,600	142	
9:00 p.m.	4,200	5,670	0.5	35		37	3,400	68	
10:00 p.m.	3,800	5,130	1.0	38		41	3,700	86	
11:00 p.m.	3,200	4,320	1.0	0		44	3,900	25	
11:30 p.m.	Bypass Off								
TOTAL					974				1,914

Storage (2004) = 974,000 gallons; (2014) 1,100,000
Bypass = 1,914,000 gallons

+ Stream influx begins

TABLE 7

KEARSARGE PUMP STATION
OVERFLOW
DATE: 12/23/04

Time	Q Meter (gpm)	1.35 x Meter (gpm)	Interval (hours)	Forward Flow (psi)	Forward Flow (gpm)	Bypass Q (gpm)	Σ Bypass (gallon)	Storage @ 4,500 (gallon)	Σ Storage (gallon)
11:45 AM	3,600	4,860			4,860	0	360		
11:45 AM	Bypass Opened 12 Turns								
12:00 PM	5,500	7,425	0.25	27	1,700	5,700	85,500	2,925	43,875
1 & 2 only Bypass at 6 Turns									
12:15 PM	5,000	6,750	0.50	33	2,200	4,500	135,000	2,250	67,500
1, 2 & 3									
12:45 PM	3,600	4,860	0.50	33	2,200	2,400	72,000	360	10,800
+Bypass Opened to 12 Turns									
12:50 PM	5,800	7,830	0.50	33	2,200	5,600	168,000	3,330	94,900
1:05 PM	Bypass Closed to 6 Turns								
1:05 PM	No Recording			38	3,000				
1:20 PM		6,000 *	0.50	38	3,000	3,000	90,000	1,500	45,000
1:40 PM	Bypass at 4 Turns								
1:45 PM	3,500	4,725	0.25	46	3,400	1,300	19,500	225	3,400
2:00 PM	3,500	4,725	0.25	46	3,400	1,300	19,500	225	3,400
2:45 PM	3,500	4,725	0.25	46	3,400	1,300	19,500	225	3,400
3:00 PM	3,000	4,050	0.25	46	3,400	600	9,000	0	
Bypass Closed									
4:00 PM	2,800	3,750			3,750	0		0	
TOTAL							618,000		272,275

* Extrapolated
+ Believe stream impact began

Note: 11:45 AM suction head at ______ (3rd step above 2nd landing)
3:00 PM suction head normal

kearsargepumpstationoverflow12-23-04Table 7.xl

TABLE 8

KEARSARGE PUMP STATION
OVERFLOW
DATE: 12/31/04

Time	Q Meter (gpm)	1.35 x Meter (gpm)	Interval (hours)	Forward Flow (psi)	Forward Flow (gpm)	Bypass Q (gpm)	Σ Bypass (gallon)	Storage @ 4,500 (gallon)	Σ Storage (gallon)
8:45 AM	3,500	4,725			4,725	0		225	
8:50 AM	Bypass Opened 6 Turns								
9:00 AM	4,500	6,075	0.50	40	3,400	2,675	80,250	1,575	47,250
9:30 AM	3,500	4,725	0.50		3,400	2,325	69,750	225	6,750
10:00 AM	4,400	5,940	0.25		3,400	2,540	38,100	1,440	21,600
10:15 AM	4,300	5,800	0.25		3,400	2,400	36,000	1,300	19,500
+10:30:00 AM	Bypass to 10 Turns								
10:35 AM	5,500	7,425		32	2,000	5,425		2,925	
10:40 AM	5,000	6,750	1.50		2,000	4,750	427,560	2,250	202,500
No Meter									
11:50 AM	5,000			32	2,000				
Lost #2 Pump									
11:55 AM	4,500	6,075	2.25	30	1,800	4,275	577,125	1,575	212,625
Noon	Meter Off								
12:10 PM	4,600	6,210		30	1,800	4,410		1,710	
Meter Off									
2:15 PM	4,500	6,075	0.75			4,275	192,375	1,575	70,875
3:00 PM	4,000*	5,400	1.00			3,600	216,000	900	54,000
4:00 PM	3,700	5,000	2.00			3,200	384,000	500	60,000
6:00 PM	3,000	4,050	1.00			2,250	135,000	0	
7:00 PM	Bypass Off								
	3,000	4,050			4,050	0			
8:00 PM	2,700	3,645			3,645	0			
TOTAL							2,156,100		700,100

* Found stream backing into station plug dislodged
\+ Believe stream impact began (10:30 a.m.)

Note: At 8:45 AM wet well 1st step above 2nd landing

kearsargepumpstationoverflow12-31-04Table 8.xl

The discharge at different heads then is directly proportional the ratio of the square roots of the heads (from the orifice equation $q = CA\sqrt{2gh}$). The difference between flows at 1.5 ft. and flows at 5.5 ft. (head on September 9th) equals ($\sqrt{5.5} / \sqrt{1.5}$) 1.9. Flows at 5.5 ft. then would equal 1,900 gpm.

Other Observations

Erie Connection (Manor Drive)

An alternative to storage is forward pumping. Figure 2 gives the results of an assessment of the flow rate increases that may be available at the Manor Drive (Boyer) connection during the September 9th storm event to enable the forward pumping capabilities at the Kearsarge pump station to be increased.

The figure demonstrates that (with the possible exception of the 2:40 a.m. flow) no flow would have exceeded the permitted total of 30.5 MGD if flows from Kearsarge had equaled 4,500 gpm. However, the 30.5 MGD includes Fairview's and Summit's allocations.

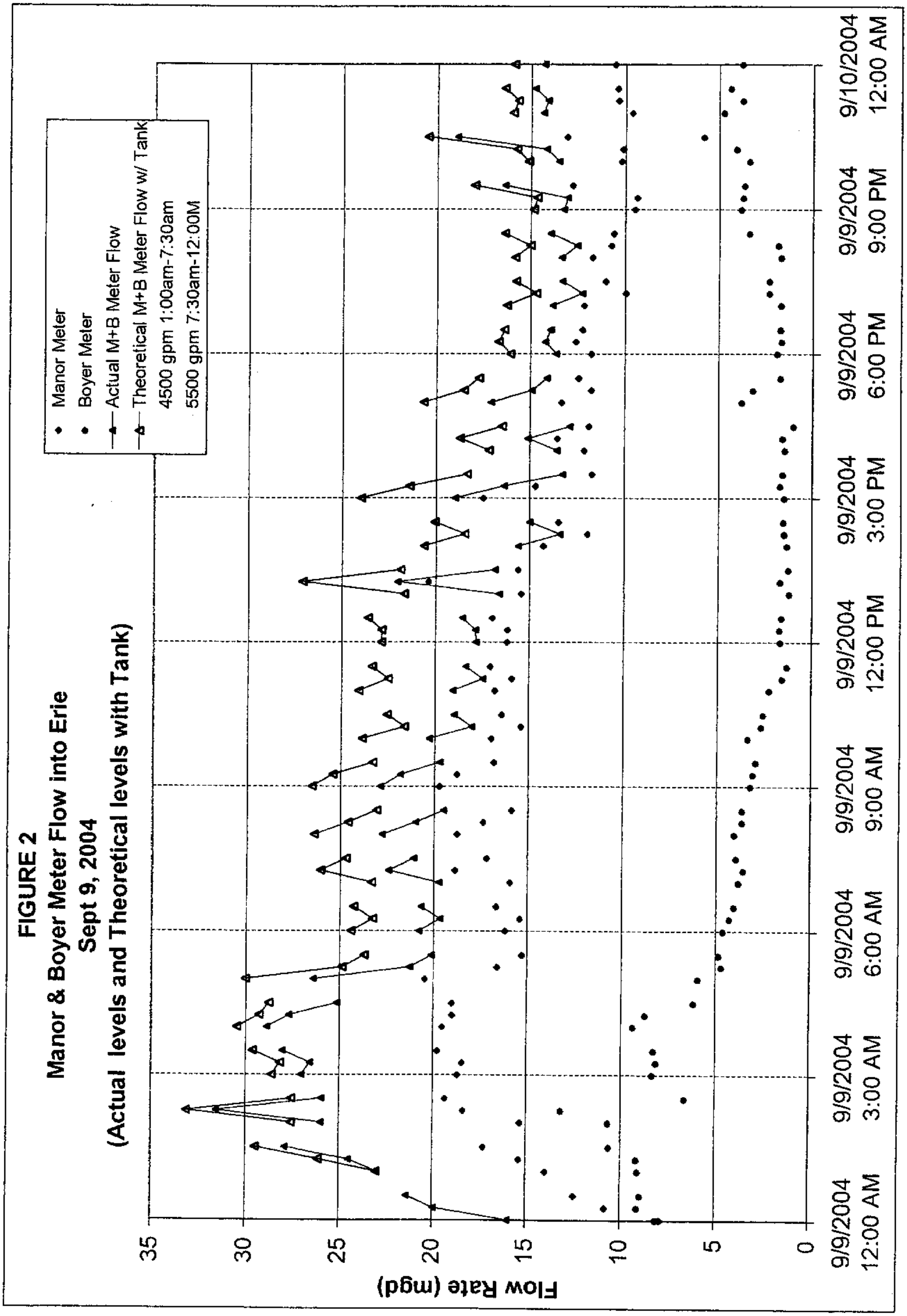
Millcreek's allocation at Manor is 23.27 MGD. Figure 3 gives the calculated Millcreek flow at that point during the rain event. That flow was obtained by deducting the Fairview and Summit flows during the event. Millcreek in this scenario exceeded its agreement amounts by a maximum of 3 MGD during the period 2:00 a.m. through 4:00 a.m. Increasing discharge flows to 4,500 gpm would have increased that duration to 1:00 a.m. to 5:00 a.m. and would have increased the peak to 27.5 MGD. After that time, however, no exceedances would have been recorded.

In fact, the figure demonstrates that no capacity surcharge would have occurred if forward flows had been increased to 5,500 gpm beginning at 9:30 a.m. until 12:00 midnight when the bypass ended.

There is the possibility that the township flow limitation may be increased. Negotiations with the City could allow the township to deduct the Erie Water Work's peak flows from their actual flows. That figure is approximately 2.6 MGD, 1,800 gpm. There is also the possibility of sharing in the Summit allocation which presently is 3.9 MGD which would allow an additional flow increment that would be useable by Millcreek of 700 gpm.

Until those possibilities are resolved, a potential alternative to reduce storage volumes needed for only the most extreme storms would be to increase the Kearsarge pumping rates toward the end of the storm as the tank fills and flows to the City decrease. This could have been done and still remain within the Millcreek allocation for the September 9th storm. This would minimize the size of the storage needed and still manage the most extreme event. As demonstrated in Figure 2 and Table 9, storage requirements could be reduced from 2,142,000 to 1,800,000 gallons for the September 9th storm (after adjustment for stream inflows are eliminated) by increasing forward flows to 5,500 gpm, 9.5 hours into the event (11:00 AM). In fact, forward flows could have actually been set

FIGURE 2
Manor & Boyer Meter Flow into Erie
Sept 9, 2004
(Actual levels and Theoretical levels with Tank)



/MTSA/Manor_Boyer/Manor_Boyer_Oflow Sept 2004.xls

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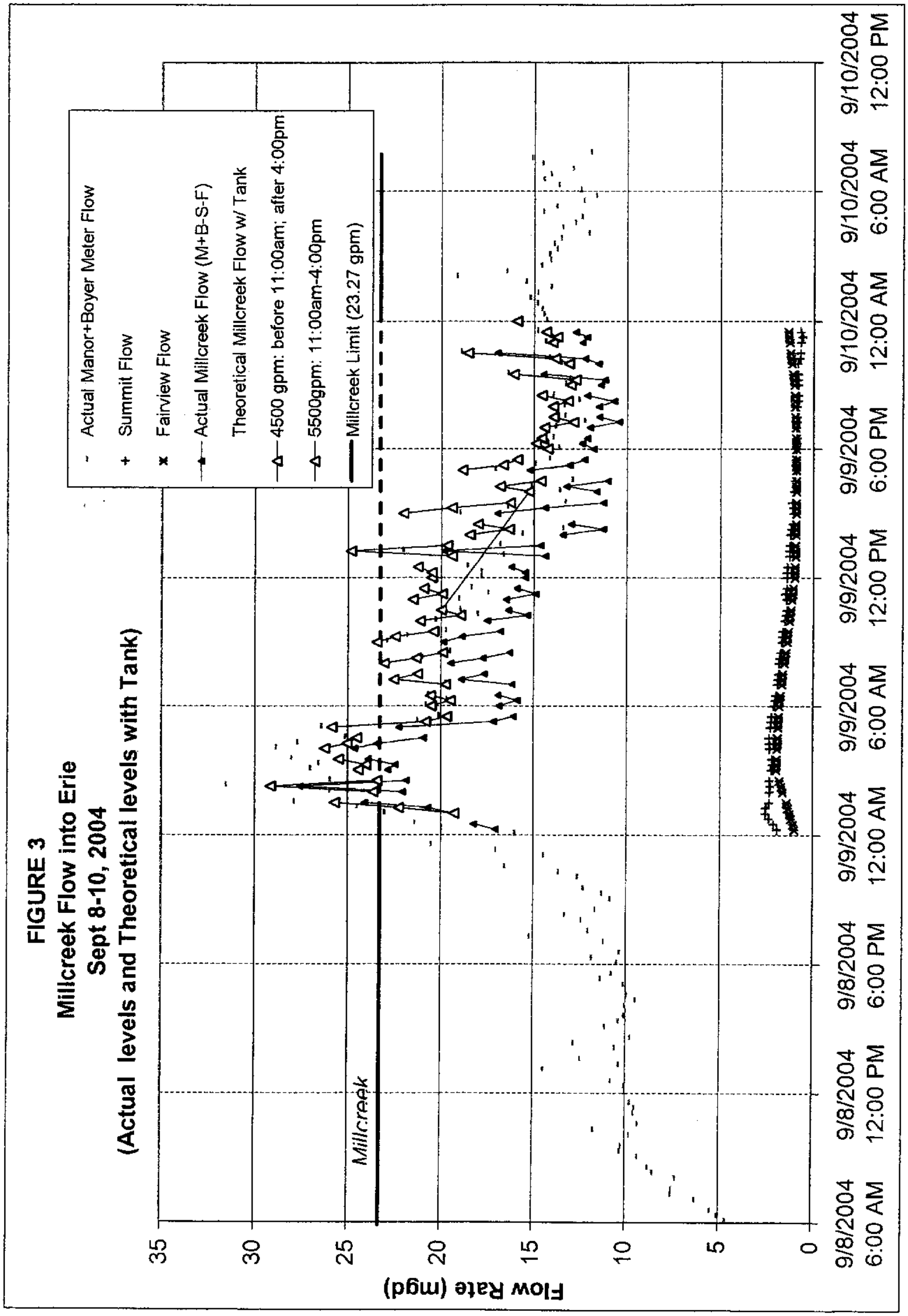

/MTSA/Manor_Boyer/Manor_Boyer_Oflow Sept2004.xls

TABLE 9

KEARSARGE PUMP STATION
OVERFLOW
DATE: 9/9/04

ADJUSTING FOR STREAM INFLUX
Forward Flow 4,500 GPM (12 Midnight - 11:00 a.m.) & 5,500 GPM (11:00 a.m. - 5:00 p.m.)

Time	Q Meter (gpm)	1.35 x Meter (gpm)	Interval (hours)	Forward Flow (gpm)	Stream Influx (gpm)	Manhole Pumps Bypass (gal)	Storage (gpm)	Σ Storage (gallon)
12:00 M	3,200	4,320		4,500			0	
12:30 AM	3,800	5,130	(0.50)	4,500			630	18,900
1:00 AM	4,000	5,400	(0.50)	4,500			900	67,500
2:15 AM	4,000	5,400	(1.25)	4,500			900	
2:15 AM	5,300	7,155		4,500			2,655	119,475
3:00 AM	5,300	7,155	(0.75)	4,500		600	3,255	
3:30 AM	5,300	7,155	(1.00)	4,500	2,000			139,200
4:00 AM	5,200	7,020		4,500	2,000	800	1,320	
4:15 AM	6,700	9,045	(0.25)	4,500	2,000	1,400	3,945	59,175
4:30 AM	5,900	7,965	(0.25)	4,500	2,000	1400	2,865	42,975
5:00 AM	6,500	8,775	(0.50)	4,500	2,000	1400	3,675	110,250
6:00 AM	6,500	8,775	(1.00)	4,500	2,000	1,400	3,675	220,500
7:00 AM	6,500	8,775	(1.00)	4,500	2,000	1,400	3,675	220,500
8:00 AM	6,700	9,045	(1.00)	4,500	2,000	1400	3,945	220,500
9:00 AM	6,400	8,640	(1.00)	4,500	2,000	1,400	3,540	230,700
10:00 AM	6,300	8,505	(1.00)	4,500	2,000	1,400	3,405	212,400
10:30 AM	6,000	8,100	(0.50)	4,500	2,000	1,400	3,000	90,000
11:00 AM	4,500	6,075	(0.50)	5,500	2,000	1,200	0	24,000
12:00 N	4,500	6,075	(1.00)	5,500	1,660	1000	0	0
1:00 PM	4,500	6,075	(1.00)	5,500	1,200	800	200	12,000
2:00 PM	4,500	6,075	(1.00)	5,500	1,000	600	200	12,000
3:00 PM	4,500	6,075	(1.00)	5,500	950	400	0	0
4:00 PM	4,000	5,400	(1.00)	5,500	900	200	0	0
5:00 PM	3,500	4,725		4,500	850	0	0	
6:00 PM	3,500	4,725		4,500	800	0	0	
12:00 M					500			
12:00 N					0			
For 2014 flows add 10,000 gpd/hr. = @16 hours = (160,000 gallon)								1,800,000 160,000 1,960,000

kearsargepumpstationoverflow9-9-04Table 9.xl

11c

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at 5,500 gpm at 9:30 AM and still remain within Millcreek's current agreement amounts thereby reducing needed tank storage by an additional 90,000 gpd. The additional 1,000 gpm pumpage rate would also serve as a contingency in the event of an even greater event or equipment malfunction and also could be used to increase the capacity of the station after year 10 or if negotiations allow, earlier. Adjusting for growth the 2014 tank size needs equals 2,000,000 gallons.

Sewer Meter K3-Beaver Run Relief Interceptor

When the past data was all corrected (one year) and was then compared to the Summit data for the same time period, it was discovered then that actual values compared much better than they had in the past. Little if any difference between the two meters were found when both were functioning properly. Correcting the data makes no difference in the previous recommendations regarding Summit flows since the Summit data was used when there were significant differences because its accuracy originally was believed greater than the inline sewer data and the more recent data confirms that belief.

Discharge Meter Calculation

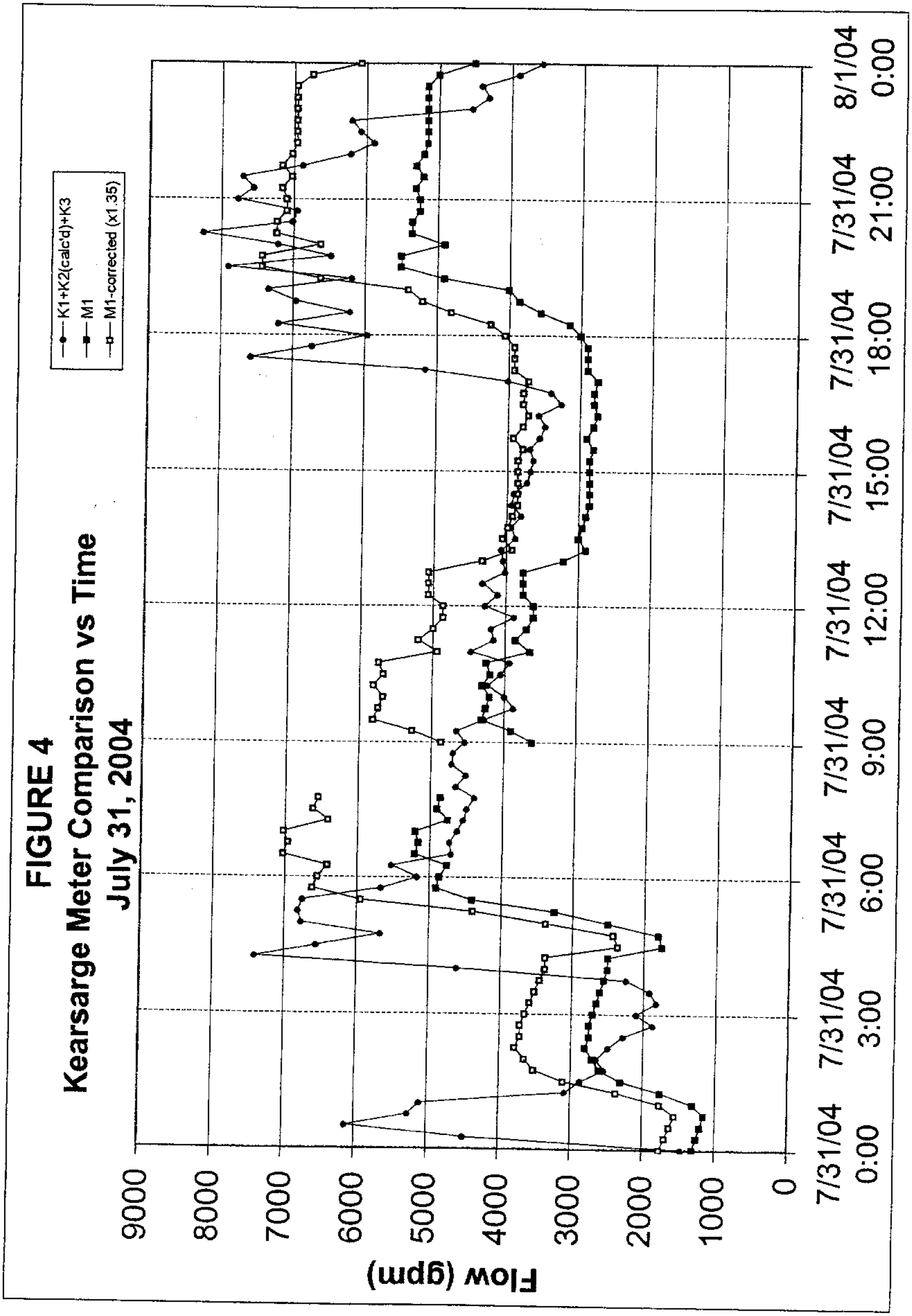
The values obtained from the most recent drawdown test compared most favorably with the pump curves. As a result it is concluded that the 135% figure obtained through the drawdown tests is correct. All results included in this report use that value.

Bypass Plug Dislodgement

The impact of the inflow through the abandoned upstream bypass on the overflow events prior to December 31, 2004, was investigated. As stated previously, the only event in which the upstream meters functioned well enough to be dependable was the July 31st storm. The upstream meters K_1, K_2, and K_3 were compared to the pump station meter for that date.

The comparison is graphed on Figure 4. There is a time discrepancy but when that is adjusted, the flows compare unusually well. It is concluded that there was no impact on flows during that storm due to the stream cross connection and, therefore, it was also concluded that the abandoned bypass plug was in place for that storm and therefore all previous storms (however, no stream level observations were made that would confirm the latter conclusion, i.e. was stream flow levels above the abandoned overflow pipe).

The September 9th and September 17th storms were both noted to have flow idiosyncrasies. Both had unexplained flow incremental increases during the event and both were noted to continue long past the normal duration previously observed after a rainfall event. Some of the September 9th unexplained flows were originally attributed to stream inflow into open manholes and other unknown sources. It is now believed they were due to stream flows entering the pump station through the abandoned overflow.



/MTSA/Kearsarge PS/Flow Data/Kearsarge Meters 2004 7July.xls

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It is postulated that the plug in the bypass was dislodged at 3:45 AM on September 9th when it was noted that the wastewater level in the wet well which had been receding began filling and it became necessary to open the force main bypass further to increase pumping capacity. It was shortly after that that stream waters were noted to have crested above the 500-year flood level (5:00 AM) due to the downstream blockage (tree and bridge abutment). Further pictures taken the following day showed stream levels still above the bypass pipe outfall.

It is known that the stream also crested above the abandoned overflow wing wall on September 17th.

CONCLUSION

Overflow Events

<u>July 31, 2004 Storm Event</u>

This storm event very much approximated a larger event discussed in the initial report. There were three separate high intensity periods observed in the drainage area (some rain intensities were not observed along the lake shore). These events occurred at midnight; 5:00 a.m.; and again at 6:00 p.m. There was intermittent rainfall with intensities less than 0.1-inch/hour before and between peaks. The peaks approximated 1-year events. The combination of the peaks equaled a 10-year frequency. The calculated storage capacity needed to accept flows from the storm in 2014 was 1,525,000 gallons and a storage pumping rate of 2,700 gpm.

<u>September 9, 2004 Storm</u>

This storm was concluded to be a 50-year twelve-hour event with a runoff period of approximately 23 hours. The observed pump station flows during this storm were impacted by a downed tree and collapsed bridge abutment downstream which flooded the area around the station. The obvious impact of that incident has been ignored in the calculations of the storage volume and storage pumping capacities.

The contributions from the stream influx for both storms were based on unexplained increases in observed flows as long as they remained within the range of calculated potential stream influx volumes.

Table 5 is an adjustment of Table 4 (September 9, 2004 Storage Flow Needs) deducting for the estimated impact of the influx of stream waters beginning at 3:45 AM when the unexplained influx of flow was first noted at the station (although pumpage was not increased until 4:15 AM). Influx flows were assumed to continue beyond midnight when the bypass was discontinued, although at a reduced rate beginning at 10:30 AM. The influx rate was reduced at 10:30 AM and assumed to remain constant until 3:00 PM because the head on the overflow was believed to be kept constant by the damming

action of the felled tree and bridge abutment. At that point it is believed the flows dropped below the tree allowing the levels to drop steadily as the stream receded until 12:00 noon when it is assumed the stream influx stopped. A picture at 10:00 AM (Appendix C) shows the stream level about midway on the abandoned outfall wing wall.

With the above assumptions the calculated needed storage volume for that storm with a forward flow of 4,500 gpm was 2,142,000 gallons. Corrected to 2014 anticipated flows to the pump station, needed storage equaled 2,303,000 million gallons. The needed peak storage pumping in 2014 (deducting the flow rates credited to the stream backup but accounting for manhole pumping) is 4,540 gpm. No storm of similar magnitude was available for the original report.

September 17, 2004 Storm

This storm would not normally be anticipated to have caused an overflow except for saturated conditions. The rainfall is believed to have totaled approximately 1.8-inches in a twelve-hour period, although one rain gauge indicates greater quantities. A similar storm observed a month later on October 15th and 16th but only with half of the intensity, 0.9-inches in twelve hours, had little or no impact on the station's pumping rate. The rainfall had been relatively steady beginning on Friday, October 15, accounting for approximately 1.1-inches by 4:00 p.m. on a Saturday. Over the next twelve hours the rainfall reached .9-inches but pump station flows only reached 2,700 gpm and that was by noon. Rain had essentially stopped by 7:00 a.m. although light rainfall did continue with another .3-inch observed by 4:00 p.m. Yet at 7:30 the station's pumping was only 1,500 gpm. It increased by 675 gpm to 2,175 gpm by 9:00 a.m. but much of that could be credited to increased domestic contribution as water usage increased in the system as people awakened.

The September 17 storm is not given credit as having any major frequency occurrence but it did create an overflow. The event was somewhat complicated as discussed previously due to power failures during the course of recovery. It was calculated that this event would have required 640,000 gallons of storage in 2014 and a storage pumping rate of 3,000 gpm.

Special Study Addendum Alternatives

Alternates to the recommendations of the original "Special Study" report to accommodate the later storms include increasing storage capacity and/or increasing forward pumping capacity or a combination of both. The alternates are discussed in the following paragraphs and all assume the September 9th storm is the design storm.

1. Maintain forward pumping capacity of 4,500 gpm and construct storage sufficient to handle the design storm. This could require construction of a 2.3 million gallon tank in 2014 (if the September 9 storm were selected as the design event) which would require the use of a 146 foot diameter tank, 25 foot side wall height, 26 foot high dome, total

height 50 ft. plus. Such a size tank requires purchase of additional land between the Authority or the township property and the stream. The tank can be constructed and remain outside the 100-year flood plain and maintain zoning distances except for height. However, it would require the removal of two large trees which are presently protected by covenant and would hinder public use of the area. The largest diameter tank usable without endangering the covenants or infringing on the 100-year flood plain is approximately 100 ft. which allows for 2.1 MG tank.

2. Increase forward pumping capacity to handle the increment over that provided by the study. In the worst case scenario this would require an additional forward pumping capacity of 3,140 gpm or 4.5 MGD. The assessment of the capacity available to the Authority for additional flows at the Manor connection at high storm flows would not permit this amount of forward flow increase at this time even with correction for Erie Water Works discharge and use of Summit Township's unused capacity.

3. Increase storage and increase forward pumping capacity during the entire storm event. This could only be accomplished if Millcreek obtained additional capacity allowances. Some variations could be implemented in this instance if the resolution of the Erie Water Works flow responsibility is resolved in Millcreek's favor. However, this cannot presently be assured and is not presently implementable. The capacity available for any continued growth in Millcreek would be depleted. Capacity obtained from Summit would only be an interim solution until Summit's growth demanded its use.

4. This alternate uses storage to handle the initial flow and would increase forward flow but only after the main storm impact had passed the interconnection and when storage capacity reached its limits. The alternate would use the storage facility during high-flow periods into the City and increase forward flow once flow had decreased into the City. The forward capacity increase could be selected at any amount within the limits of the pumping capability and could be controlled by the SCADA system.

RECOMMENDATION

Event Selection

Of the events that have been observed to this point both in the original report and since July of 2004, it is believed that the July 31, 2004 storm, a 10-year event, is the most reasonable design event but with facilities sized to allow accommodation of larger events such as the September 9th event. This latter event was between a 30 and 60-year storm event (believed to be a 50-year event). Even a 50-year event causes the construction of a large size storage facility which will remain unused for long periods of time. Any greater event is believed beyond the predictive accuracies of any design. Instead, contingencies on how additional flows may be handled and how damages can be minimized (i.e. use of standby equipment and use of sewer line storage using backwater preventers to prevent user damages) are proposed.

Concept Selection

The optimum alternate which would minimize the footprint of the tank and make maximum use of facilities would be a tank sized based upon maximum utilization of the Authorities agreed upon flow limits into the City (Alternate 4).

That concept will allow peak forward flows of 5,500 gpm to be achieved used toward the end of the storm runoff period. On September 9th the overflow waters would have been stored until the forward flow could be increased while still maintaining Millcreek flows into the City within the agreement amounts. Flows from the station could have been increased to 5,500 gpm on September 9th beginning at 9:30 AM. Table 8 shows the impact of that change on the required storage if forward flows had been increased to 5,500 gpm at 11:00 AM. Storage required would have equaled 1.8 MG initially. Increasing the storage size to handle 2014 flows equals 2.0 MG.

However, it has been determined that the station forward pumps could not achieve the desired forward flow of 5,500 gpm. Therefore, it is concluded that Alternate 1 is the best implementable alternate. This requires a 2.3 MG tank which has a diameter of 112 ft. and a height of 33 ft. (32 ft. water depth).

With this design the control system will limit the pump speeds to maintain forward flows no greater than 4,500 gpm. When station influent flows exceed that amount, they will overflow into a pump station to be transferred to a storage tank. The overflow level will be selected to prevent basement flooding and manhole overflows but will allow some surcharge of the influent sewers (5 ft.).

The control system will allow the forward pumps to ramp up to full speed as a contingency in the event the storage pumps do not start or the tank fills. That speed will produce an additional flow increment equal to 500 gpm. Back flow preventers will be placed on all homes that have a potential for basement flooding as an additional precaution in the event of an extreme event or system failure where sewer storage is needed (September 9th demonstrated the possible impact of a downstream obstruction allowing flood waters to enter the station directly). Finally, operators will have the capability of manually operating standby pumps for an even greater contingency.

Not to be ignored is the sewer Authority and township's very real efforts to reduce inflow from basement storm water sumps. Much of the residual overflow found in the September 9th storm is believed attributable directly to those facilities.

To summarize, a 2.3 MG storage tank with a 4,500 gpm forward pumping capability will meet the 50-year design frequency event needs. Peak forward flow capabilities of 5,000 gpm will provide a contingency to address even larger events. Greater events may require the use of other operational alternatives not normally considered acceptable but preferable to environmental and health damages.

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Tank Selection

The required dimensions of the tank and its location on the property is dictated by the design storage requirements and environmental, esthetic, and legal restrictions. The minimum tank size needed has been selected to minimize environmental impacts such as esthetics and tree removal. The siting has been dictated by separation requirements from utilities (electric) and flood plains, wetlands, and by esthetics (provide the least visibility).

Figure 5 shows the site plan locating the tank and its supporting structures. Figure 5 locates the tank to keep it out of the 100-year floodplain and to make the most use possible of the purchased land thus avoiding as much as possible conflict with the other lands' uses. Public access to the area is maintained, but two large sycamore trees have been unable to be protected. The earlier smaller tank preserved the larger of the two trees. However, when it was determined a larger tank was necessary, the siting had to be reevaluated. Initially it was hoped that the larger tank (dictated by the September 9th storm) siting could be manipulated through purchase of additional land and minimizing tank diameter. Those efforts were unsuccessful and the larger tank's footprint infringes upon the root system for both trees. It must be assumed they will be lost.

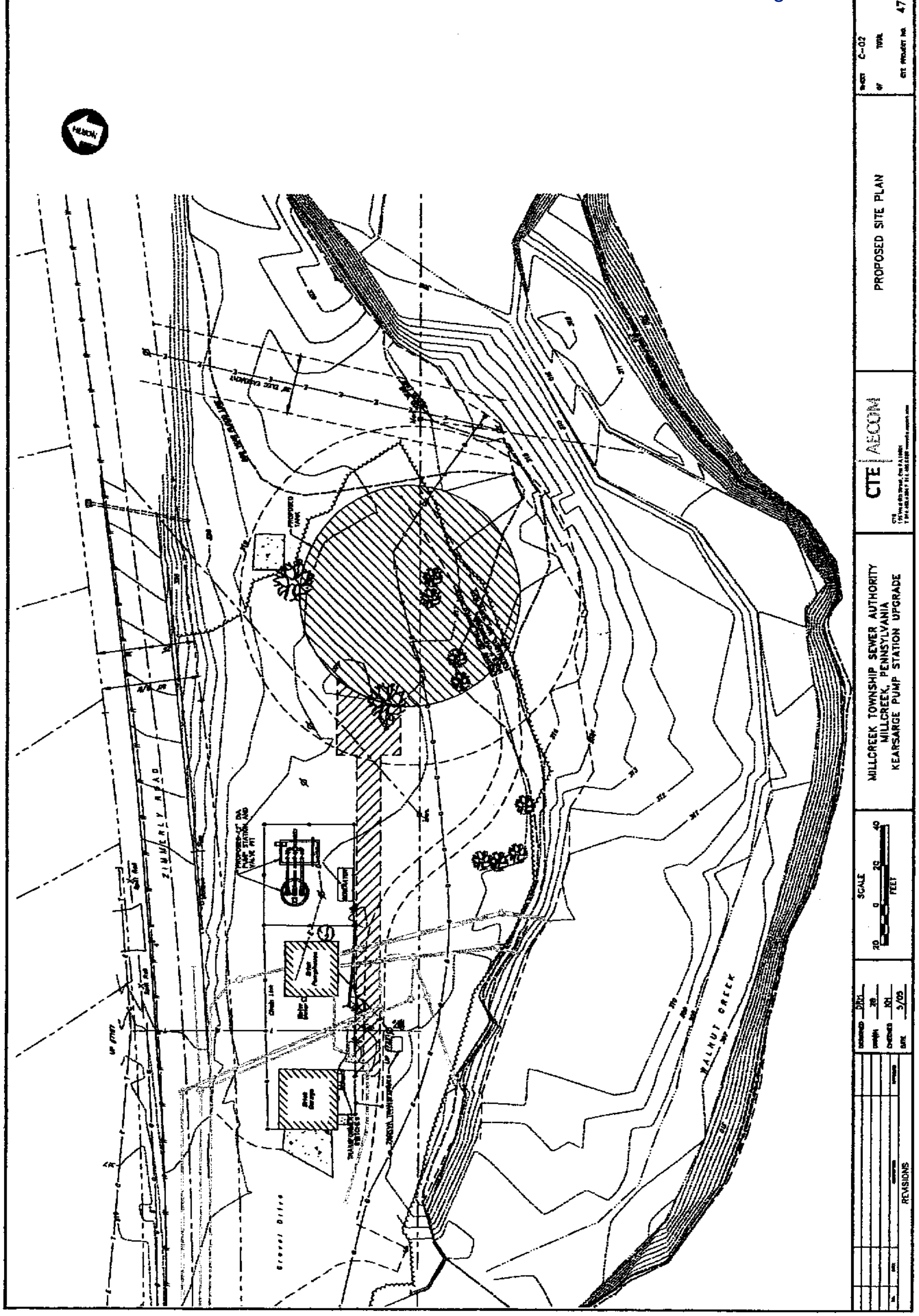
Presently many overflow retention tanks are being installed in Pennsylvania without covers. Disadvantages of such a concept are potentially uncontrollable odor emissions; loss of capacity or inflow due to rainfall directly into the tank (a 1-inch rainfall in a 100 ft. diameter tank equals 5,000 gallons); leaves and debris entering the tank (trees are greater than 70 ft. high); and security and safety issues. A disadvantage to the roof is concern over confined spaces, costs, and height addition (zoning).

A tour was arranged to observe and discuss problems of similar tanks constructed in Pittsburgh. Of primary concern was odors and maintenance. No problems were reported with odor, although contingencies were provided (air or hydrogen peroxide). This installation will allow for use of the latter. A report on the tank visitation is included in Appendix F.

"SPECIAL STUDY" PLAN CONFORMANCE

It is believed the alternate changes discussed here do comply with the original study's recommendations. That study as approved stated that "observations of additional overflow events will be evaluated during the time frame between completion of this report and completion of design to assure the validity of design." This report does that.

The initial study also states on page 47 (Appendix E) that "The main reason to minimize the sizing is to minimize the environmental impact of the structure itself. If the greater good is serviced by a larger tank, it will be installed." This report demonstrates a larger tank is necessary and relocates the unit to minimize environmental damage. However, while the initial plan only took one of the large trees, this plan will take both.


PROPOSED SITE PLAN
MILLCREEK TOWNSHIP SEWER AUTHORITY
MILLCREEK, PENNSYLVANIA
KEARSARGE PUMP STATION UPGRADE
CTE AECOM
SCALE
FEET
C-02
47127
REVISIONS
ZIMMERLY ROAD
WALNUT CREEK
Gravel Drive
PROPOSED TANK
NORTH

Finally, the initial study stated on page 46 (Appendix E) "minimizing aesthetic impact to the extent possible (location, height, etc.); and allowing for all pumps to operate together to allow for increased forward and storage flow to address possible extreme weather conditions," which we believe allows for the proposal to increase forward flows in the event available storage is threatened.

The plan changes are enumerated below.

a) A 2.3 MG tank is to be provided vs. a 0.5 MG unit

b) A 50-year overflow event is to be used rather than a 1-year event.

c) An automatic contingency is included to increase forward flows if tank volumes approach capacity during the end of the storm impact when City tributary flows have decreased below agreement amounts.

d) The two large sycamore trees will need to be removed rather than just the single unit originally proposed.

e) The tank will not be covered with a 212 ft. diameter unit with a 33 ft. side wall vs. the original proposal for an 80 ft. diameter unit with a 15 ft. side wall (expandable to 30 ft.) and a 10 ft. high domed roof.

f) Odor control chemical feed equipment will be designed in the unlikely event they are needed.

APPENDICES

APPENDIX A

APPENDIX A

KEARSARGE PUMP STATION
FLOW METER DRAWDOWN TESTS

September 30, 2004

Test between Low Level = Top of Fillet = 295.0 ft.
High Level = Top of Beam = 299.0 ft.
Volume = 6161.3 gallons (Revised 10/6/04)

FILLS:

#	TIME TO REFILL	WW GPM
1	3:55.31	1571.0
2	3:46.06	1635.3
3	3:31.06	1751.5
4	3:48.72	1616.3
5	3:48.60	1617.1
6	3:48.41	1618.5

AVG = 1635.0 GPM

DRAWDOWN:

#	PUMP(S)	TIME	WW GPM	PUMP GPM	AVG METER GPM	PUMP Q METER Q
1	1	3:58.32	-1551.2	3122.2	2247.4	1.39
2	1	4:01.44	-1531.1	3166.4	2302.6	1.38
3	2	5:50.06	-1056.0	2807.5	1979.2	1.42
4	2	5:52.16	-1049.7	2666.0	2008.9	1.33
5	1 & 2	2:35.56	-2376.4	3993.5	2911.3	1.37
6	1 & 2	2:36.69	-2359.3	3977.8	2903.2	1.37

AVG = 1.38

APPENDIX B

HISTORY

SEPTEMBER 8 – 9, 2004 STORM

KEARSARGE PUMP STATION

Date	Time	
9/8	5:00 p.m.	Rain started
9/8	11:00 p.m.	1-inch rain
9/9	12:30 a.m.	Wet well filling
9/9	1:00 a.m.	Gary receives call
	1:30 a.m.	Arrive station
		Wet well level 315 (4 steps down)
		Q = 4,000 gpm
		Power failure
	2:00 a.m.	Power restored (main breaker tripped)
		Highest wet well level 316.87 (1 step down)
		GCA notified
		Additional 2-inches rain
		Open bypass 9 turns
		Q = 5,400 gpm
	2:45 a.m.	Q = 5,400 gpm
		Wet well @ 311.9 (9 steps down)
		P = 30-41 psi
	3:00 a.m.	Q = 5,400 gpm
		Pumps being dispersed
		52nd & Zimmerly operating
		Tree and bridge abutment noted gone
	3:15 a.m.	Q = 5,400 gpm
		10 steps down (311.25)
	3:30 a.m.	Pump at Patton
		Pump to Larchmont
	3:45 a.m.	Q = 5,400 gpm
		Wet well above 10th step
	4:00 a.m.	Wet well above 9th step (312.25)
	4:15 a.m.	Q = 6,700 gpm
		P = 31-33
		Bypass opened to 18 turns
	4:30 a.m.	Q = 5,900 gpm
		P = 35-38
		Bypass closed to 13-1/2 turns
		Wet well at 12 steps down (310)
		Complaints from Dixon

Date	Time	
9/9	4:45 a.m.	Q = 5,900 gpm Wet well above 11th step Opened bypass to 18 turns Q = 6,700 gpm P = 31-33 Wet well at 11th step (310.6)
	5:00 a.m.	Q = 6,400 gpm Wet well at third landing First noted stream above bank & above 500 year Rehab Center at 52nd Street east of Peach Street backing up – overflowing toilets Rain light to drizzle
	5:30 a.m.	Q = 6,800 gpm Wet well at third landing (no change) Stream reached gate of pump station Felled tree observed to be damming stream
	6:00 a.m.	Q = 6,900 gpm Wet well below third landing Stream receding
	6:15 a.m.	Q = 6,700 gpm Wet well below third landing
	6:30 a.m.	Creek at top of bank Construction workers arrived – stated tree and abutment in place at 4:00 p.m. of 9/8/04
	6:45 a.m.	Q = 6,500 gpm Wet well second step below third landing
	7:00 a.m.	Wet well between second step below third landing Rain stopped Additional 1.5-inches rain
	7:30 a.m.	Noted several manholes open in previously flooded areas Q = 6,700 gpm
	7:45 a.m.	Noted pump at 52nd and Zimmerly not working WL 4-inches above manhole rim Manhole at northwest corner and one west on Zimmerly discharging around lid sizes and through manhole vent holes Wet well at third step below third landing (307.5)
	8:00 a.m.	Q = 6,700 gpm 52nd and Zimmerly not running Wet well below third step End observations
	10:00 a.m.	Wet well below first landing (302.5) Q = 6,200 gpm
	10:30 a.m.	Q = 4,500 gpm (believe bypass partially closed or pump #3 stopped and pumps #1 and #2 backed off)

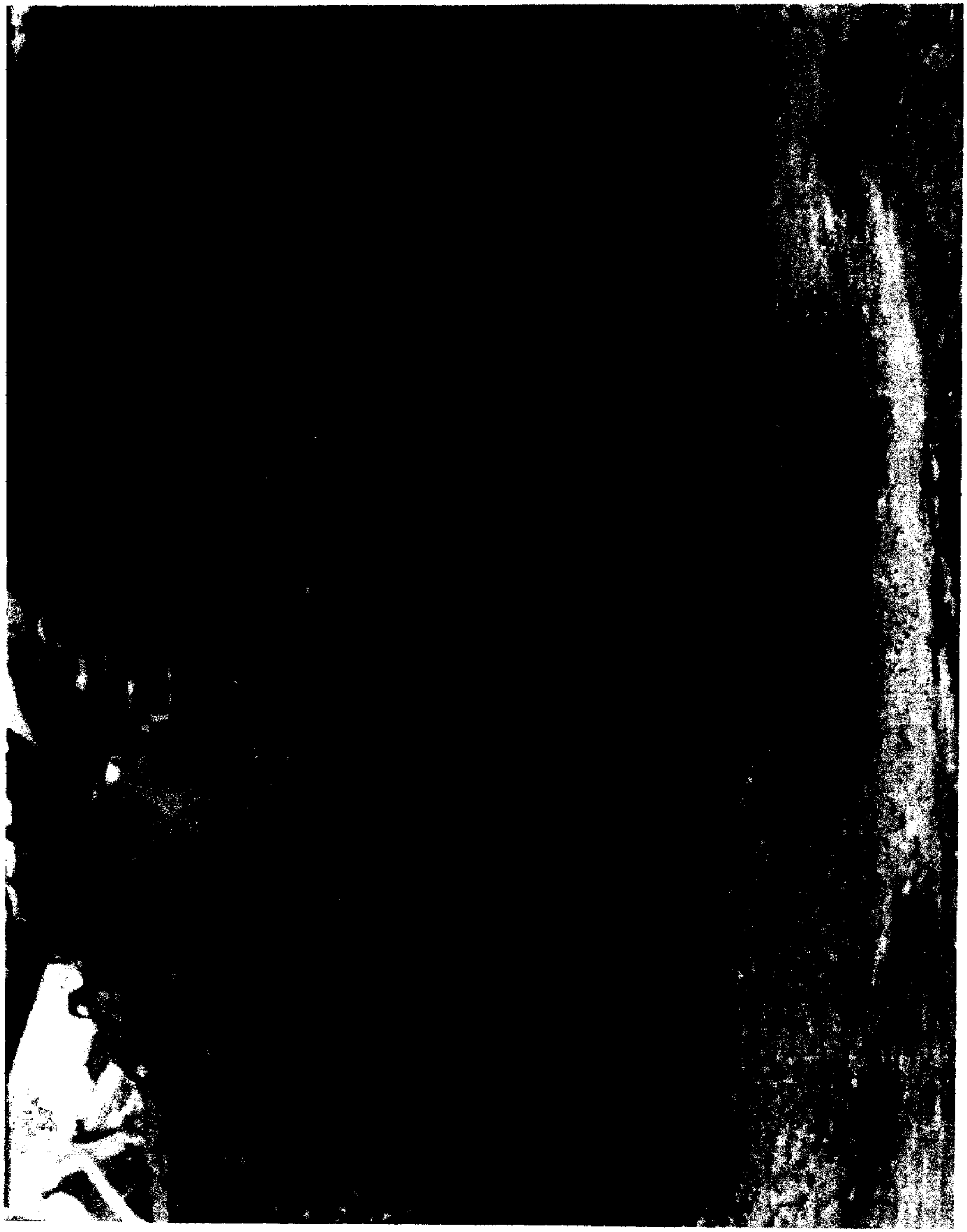
MSA-MT 2885

App. 584

Date	Time	
9/9	11:00 a.m.	Wet well at second step below third landing Q = 4,500 gpm Pumps #1 and #2 operating
	12:00 p.m.	Wet well above second step below third landing
	2:00 p.m.	Wet well at first landing Q = 4,500 gpm
	3:00 p.m.	Pumps #1 and #2 off and on
	3:30 p.m.	#1 Pump motor burned up Blew out breaker Operating #2 and #3 pumps
	4:30 p.m.	Q = 3,500 gpm Wet well at second step above first landing (guess from 10-inch levels)
	5:00 p.m.	Q = 3,560 gpm
	6:00 p.m.	Q = 3,500 gpm Wet well at first landing
	6-8:00 p.m.	Bypass off to 13 turns – Pumps removed from manholes Q = 3,500 gpm Wet well below first level
	9-10:00 p.m.	Pump #2 running Q = 3,000 gpm x 1.35 = 4,050 gpm Bypass reduced to 10 turns Pumps #2 and #3 running
9/10	12:00 a.m.	Pump #2 running Q = 2,400 gpm Closed bypass
	12:30 a.m.	Pumps #2 and #3 running Q = 2,400 gpm
	2:00 a.m.	Pump #2 running Q = 2,200 gpm

App. 585

APPENDIX C

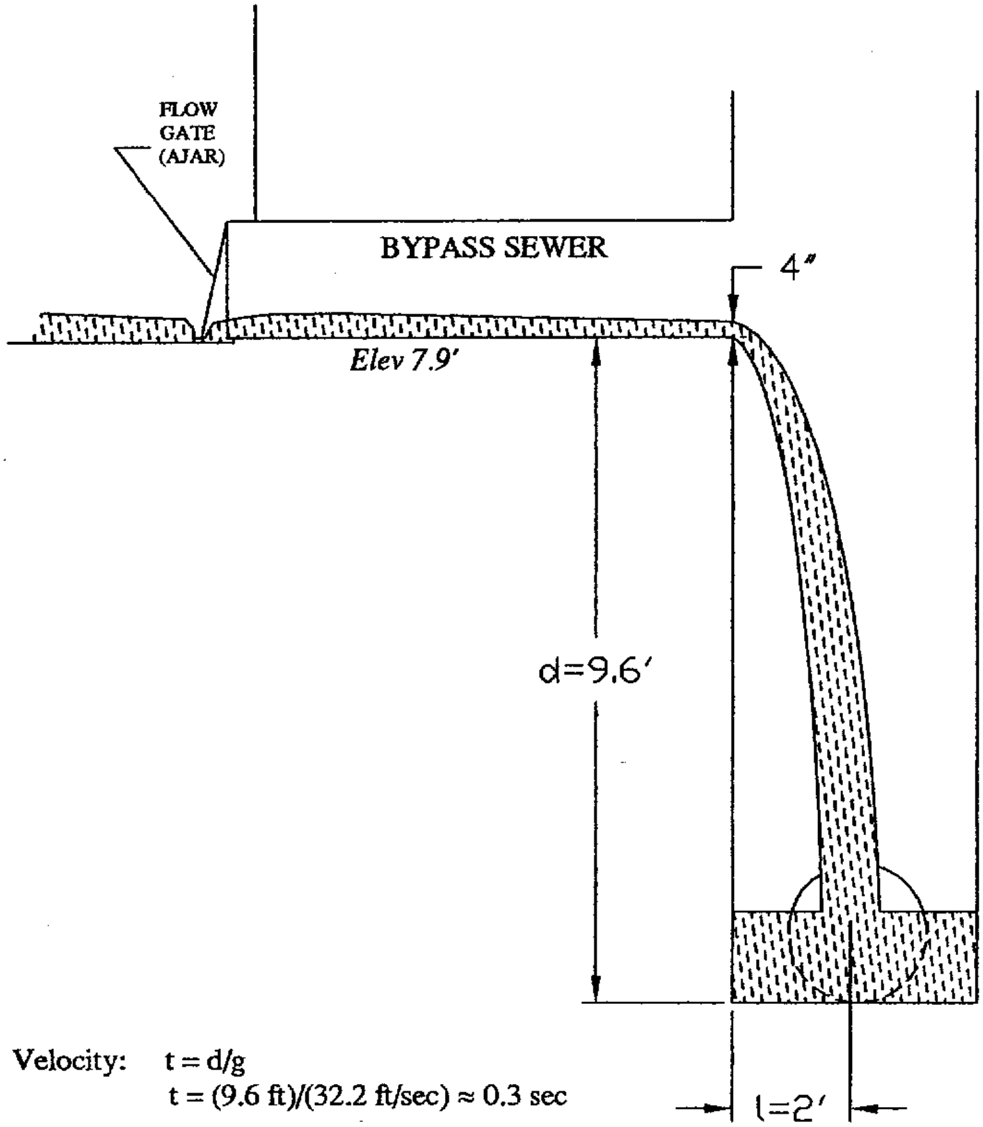


MSA-MT 2888

APPENDIX D

STREAM INFLUX FLOW ESTIMATE CALCULATIONS

DECEMBER 31, 2004



Velocity: $t = d/g$

$t = (9.6\text{ ft})/(32.2\text{ ft/sec}) \approx 0.3\text{ sec}$

$v = d/t = 2\text{ ft}/0.3\text{ sec} \approx 6.66\text{ ft/sec}$

Flow $Q = v \times A = (6.66\text{ ft/sec}) \times 0.33 = 2.2\text{ cfs}$

$= 1.42\text{ mgd}$

$= 1000\text{ gpm @ }1.5'\text{ H}$

Flow @ 5.5' head (9/9/2004 peak)

$Q_1 = CA\sqrt{2gh_1}, \quad Q_2 = CA\sqrt{2gh_2}$

where $Q_1 = 1000\text{ gpm}, \; h_1 = 1.5', \; h_2 = 5.5\text{ ft}$

$Q_2 = Q_1 \times \sqrt{h_2} / \sqrt{h_1}$

$Q_2 = Q_1 \times \sqrt{(5.5')} / \sqrt{(1.5')}$

$Q_2 = 1000 \times 2.34/1.22 = 1000 \times 1.9 = 1900\text{ gpm}$

say $Q_2 \approx 2000\text{ gpm}$

MSA-MT 2890

/KTSA/Kearsarge PS/ORF/Drawings/StreamBackFlowCalcs.dwg

APPENDIX E

under catastrophic failures; continued I&I abatement to minimize future construction requirements and provide reserve capacity in receiving sewers. Contingency recommendations include: using storage tanks where capacity can be expanded without foot print changes; investing in structural additions to allow the storage tank to be increased in height or for a second tank nearby. Minimizing aesthetic impact to the extent possible (location, height, etc.); and allowing for all pumps to operate together to allow for increased forward and storage flow to address possible extreme weather conditions.

The proposed construction includes:

1) Replacement of existing pumps with three new variable frequency controlled pumps with a capacity of 4,500 gpm forward with two pumps operating at 135 ft. total head

2) 500,000 gallon glass-coated steel tank (80 ft. diameter, 15 ft. high with dome roof, dormer, stairs, platform and water cannons for cleaning)

3) Self-contained submersible pump station, three pumps (2,500 gpm with two operating @ 30 ft. of head)

4) New generator gas operated, sized to operate three 150 Hp pumps and two 75 Hp pumps

5) Pump station upgrade features:

   a. Auto transfer switch
   b. SCADA
   c. Odor Control
   d. Ancillary features (panels, lighting, rain gage, etc.)

6) Sewers – diversion, backflow preventers, I&I abatement

B. **Justification**

1. Existing Wastewater Disposal Needs

The proposed plan will enable the elimination of the overflow and manhole pumping required by the Consent Order and Agreement and allow the waste to be pumped forward to the Erie Wastewater Treatment Plant at the proposed contingency peak flow event of a one-year storm during a wet saturated condition. It will enable service to existing residential units both in Summit and Millcreek which have been demonstrated by separate studies to be served with malfunctioning onlot systems.

MSA-MT 2892

We conclude that a conceptual design which uses as a design flow a 1-year occurrence frequency storm based on old NOAA storm frequencies * (0.9 inch/hour; 1.05 inch/2 hour; 1.22 inch/3 hour; and 2.1 inch/24-hour) during the cooler months is adequate protection during those months.

Further it has been demonstrated that such a design will more than compensate for a 50 to 25-year storm occurring in the warmer months (May through November). However, the concern is that a sustained rain during the warmer months could saturate the soils leaving the station susceptible to a higher probability of being subjected to a less frequent high intensity storm. Provision to use standby pumping facilities to address such an extreme event will be included.

Also, observations of additional overflow events will be evaluated during the time frame between completion of this report and completion of design to assure the validity of design. Cost differences for materials and erection costs for upsizing the ORF are relatively small for 500,000; 750,000; and 1,000,000 gallon tanks ($460,000; $514,000; and $618,000). The main reason to minimize the sizing is to minimize the environmental impact of the unit structure itself. If the greater good is serviced by a larger tank, it will be installed. Up to a 1,000,000 gallon tank can be installed and still meet zoning. Thus the 500,000 gallon tank called for by this report is to be considered a minimum sizing until later data confirms its validity.

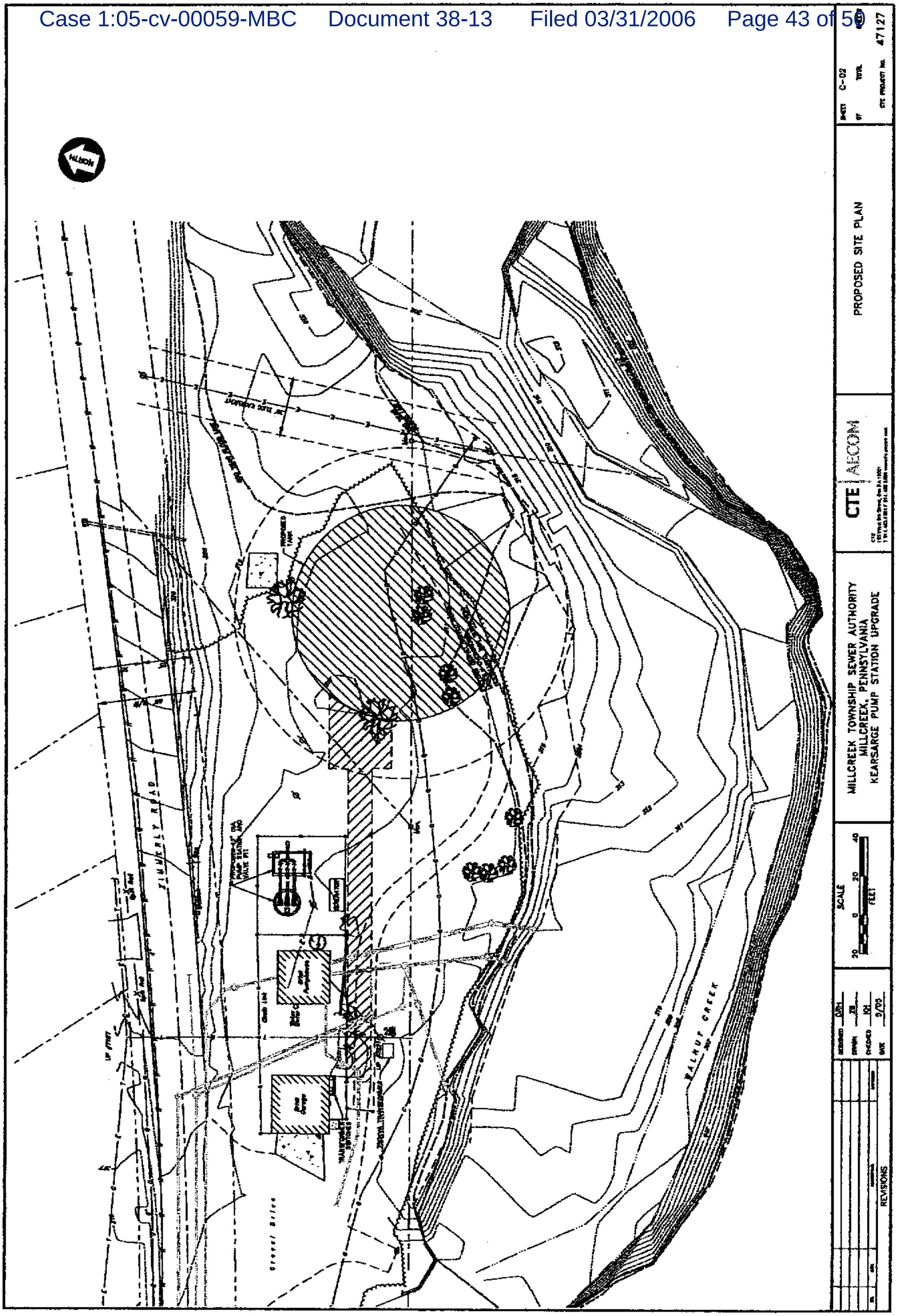
2. Future Wastewater Disposal Needs

The proposed facilities are sized to handle the ten-year growth in Millcreek and Summit and to be easily expandable to encompass the ultimate growth flows by forward pumping or storage if I&I abatement efforts do not eliminate the need.

* New NOAA frequencies do not give a 1-year frequency storm

3. Operation & Maintenance Considerations

The proposal will not increase the number of facilities needing service and it will simplify operation and maintenance by automating the emergency generator operation; eliminating the need to manually attend the station and tributary sewer manholes during high flow events (minimizing alarms); replacing aging equipment; and reduction of odors and basement flooding. It will increase power requirements, complicate control commands, and increase the number and size of facilities required to be maintained, particularly the control equipment. There should be no net change in the township maintenance employees' duties but will require more contract activity to maintain electronic and control equipment. The


NORTH
ZIMMERLY ROAD
WALNUT CREEK
Gravel Drive
REVISIONS
SCALE
20 0 20 40
FEET
MILLCREEK TOWNSHIP SEWER AUTHORITY
MILLCREEK, PENNSYLVANIA
KEARSARGE PUMP STATION UPGRADE
CTE AECOM
PROPOSED SITE PLAN
C-02
47127

APPENDIX F

CTE | AECOM

CTE
155 West 8th Street, Erie, PA 16501
T 814.453.4394 F 814.455.6596 www.cte.aecom.com

Letter of Transmittal

Date: 4-1-05
To: George Riedesel
Firm: Millcreek Twp. Sewer Auth.
Address: 3608 W. 26th St.
Erie, PA 16506
Phone No.: 835-6721
Fax No.: 835-6615

From: Jerry Allender
Phone No.: 453-4394
Fax: 455-6596
Job No.: 47127
Subject: Tank Inspections

We are sending:

☐ shop drawings ☐ specifications ☐ for your approval ☐ approved ☐ plans ☐ originals ☐ for your use ☐ as requested ☐ samples ☑ for review/comment

Copies	Date	Description
1		GCA Memo on MTSA - Overflow Retention
		Facilities Inspections - Monroeville, PA

Remarks:

Sincerely,

Gerald C Allender

cc:

File No.: 47127 - Kearsarge

We suggest that the design of the tank include wheel operated man ways at the bottom but two would be sufficient. We also recommend that the cannons be mounted on the out side of the tank and that stairs be utilized for access but we would suggest the use of the ladder for the first 8 ft. protected by the doorway mentioned earlier. The cannons will need to be relatively high pressure (100psi) and the cannons will require a booster pump for the water. The drain should be provided with a tee with a cleanout at the end of the unit. Also the drain should be vented so that when it is drained there will be air relief. We suggest no cover although my concern continues with the leaves, particularly since the two large trees are to remain in this particular instance because of environmental issues. A feed system should be allowed for hydrogen peroxide addition for odor control. (With little usage the smaller capital cost with larger operating costs is the most cost effective particularly since it is not believed necessary.) Drains should be controlled by the SCADA system and there should be a flow control valve. As stated, the Plum Borough and Penn Hills use plug valves and they only plugged when some stones and rocks got into the tank because of some sewer construction. They had to remove the valves for cleaning so the valves should be located so that it can be accessed to be cleaned.

The tank should be constructed to put a roof on if there are unmanageable problems. Odor problems are not anticipated being a problem. They are not a problem in any of these cases and some are located very close to residential areas and they have had no complaints. However, they are aerated, two by blower and one chemically. I suggest we allow for adding a chemical feed in the future if odors become a problem. The only complaint they've had thus far is that tanks which are aerated can create foaming and foam blows off into the residential neighborhoods.

If the center unit is utilized, it should be understood that the cannons cannot be turned down sufficiently to push or clean the tank to the center drain. The Plum Borough people utilize flexible hoses fastened to the end of the cannons to allow them to clean these areas.

There is some concern about safety. Plum Borough has rope ladders available to get down in the event someone should fall in. In the Plum Borough case, persons could cling to the center cage. No such facilities exist in the Penn Hills unit. The Penn Hills unit uses the wheel opening entries, but they would not be usable with a particularly full tank. Throw life preservers and a rescue chair are possibilities.

There should be protection against floating materials. We suggest the use of a goose neck at the wet well entry to the pump station as the best means of accomplishing that.

If a roof is to be provided, it will become an obstacle to cleaning and that also would have to be investigated.

GCA/lb

valves. As stated these units had water cannons at the center shaft of the unit. Pressures close to 100 lbs. are probably necessary to operate these cannons to reach the exterior walls of the unit (MTSA pressures are only 50 to 60 lbs). The tanks that we looked at were approximately 100 ft. in diameter, similar to what we are talking about in Erie. These cannons located in the middle cannot reach directly under them to the drain. They use fire hoses connected to the cannons. They also keep rope ladders on hand for rapid access. Access to the ladder on this second unit was prevented by a "U" channel shaped door that enveloped the first six to eight feet of ladder below the hoop. This tank also is provided with blowers.

The third tank at Penn Hills was a similar size and approximately 100 ft. in diameter. It, however, had access using stairs along the perimeter of the tank. Stairs were provided at 180 degrees to each other and the cannons were mounted at the top. There was no center column or scaffolding up above. These stairs cost about $700/vertical foot according to the Aqua Store people (with about a 35 ft. tank this is probably about $25,000). Mr. Riedesel did like the stairs, although the cost was a concern. I am concerned about the fact that it would be an attractive nuisance. He felt that the fence with barbed wire would take care of this. I did talk to Aqua Store about the possibility of having a ladder for the first 10 vertical ft. that it could be protected by the "U" channel door referenced in the second Plum Borough tank. The Penn Hills unit feeds hydrogen peroxide to oxygenate the waste to protect against odor.

The Penn Hills operator indicated that their fill system had originally used a flow meter that was placed on the gravity sewer but it washed out and they then changed to a level control. They use Kishner brushes to prevent floating debris from entering the tank. Floating debris is of course a potential problem with the normal type of floating materials found in sewage. These operators also indicated that their drain was not provided with a vent and as a result, when they were draining, they ended up with a lot of surging and belching of the drain. Their influent also entered at the bottom. It was not a goose neck like the two at Plum. They indicated that the inlet froze because it did not have a drain. We did discuss the drain here in the office. There were objections to providing a 1-inch or ½-inch hole to allow drainage of the inlet pipe to the ground to prevent freezing. After consideration it was thought that that hole could actually be placed in the inlet pipe back at the valve vault at the pump station. The material could then just drain from the valve pit into the wet well of the pump station and be handled at that point.

Mr. Riedesel felt comfortable with these tanks. He also felt that the stairs should be utilized. He was concerned more with the cost of the stairs than with the potential impact of persons having easy access to the top of the tank. The operation with the cannons at the side and using the stairs is probably less costly than constructing the overhead walkway and the center column or lattice work. This tank also is drained using a plug flow control valve operated by the SCADA system. Its overflow is permitted by the County Health Department and they meter that overflow and report the volumes that they overflow out the top of the tank.

only two complaints heard were from the Penn Hills people who indicated that there should be a vent provided on the drain to allow the water to drain without bubbling and burping. They also indicated that the railings of the stairs should be somehow filled with materials because there were a lot of bee nests in these facilities.

The Plum Borough unit found at their WWTP (Holiday Park treatment facility) was fed by two submersible pumps of approximately 3,000 gpm capacity. The units were located in an extension to the influent wet well and were constantly flooded. Once the operating level was reached, they would activate. When first installed the wet well was not large enough. The pumps pumped it down too quickly and would overheat and thus the wet well was expanded to double that size. Their operating time at full speed is about two minutes. The pump well is not deep and there is not a major construction effort, simply an extension of the wet well and two doors with the units suspended on rails. The electrical control panels are located in panels aboveground next to the rails. As stated before flows are pumped through a goose neck facility that runs to the top. The pumps are constant speed units. The drain is a manual operation for this particular overflow retention facility and the plug valve is opened to allow the drainage back into the plant. Control is by level in the wet well. This will be changed to operate by SCADA once a major expansion takes place this year. This unit is provided with an aeration grid and blowers for odor control. However, the operators do not see the need but they do operate it. Ladder access was prevented by a locked hinged circular plate at the start of the hoop.

The second tank at Plum Borough is located on the system by the Turnpike. That particular unit accepts flow from the interceptor when levels reach preset elevations. Pumps then discharge to the tank in the same manner as they discharge to the tank at the plant. There is an air release valve on this particular discharge goose neck. The earlier tank had been provided with entrance portals at the base that were all bolted. This particular tank had four inlets and the inlets were actually wheel operated doors similar to that found in submarines. We understand they are expensive. However, the advantage to these was given as being a more rapid entry in cases of emergencies or for the need to clean. They, of course, could not be opened for entry until the tank was drained. In this case the tank is drained back to the interceptor based upon levels in the interceptor and is controlled by SCADA and an automated flow control valve. That valve is a plug valve. When asked if there had been a maintenance problem, their engineer indicated that they had one incidence when they had stones and rocks in the facilities that had jammed the valve and they ultimately had to remove the valve to clean it. There was no other problem reported due to leaves or debris. We believe that the valve should be so located so that it is readily accessible in a valve vault and at a relatively low depth of 5 to 6 ft. It is also suggested that we use a tee at the bottom of the tank to a cleanout on the opposite side to allow the discharge line to be maintained. We talked to the engineer and the operators about leaves. They had not had leaf problems. Since we may have some rather large trees near the tank, we expect we would see numerous leaves in the fall of the season which could impact the operation of the

TO: File

FROM: Jerry Allender

DATE: March 17, 2005

RE: Millcreek Township Sewer Authority (MTSA)
Overflow Retention Facilities Inspections
Monroeville, PA

On Thursday, March 10, 2005, the writer accompanied Mr. George Riedesel, Executive Director of the MTSA on an inspection of overflow retention facilities in and about Monroeville. Two different wastewater facilities were examined: Plum Borough, Holiday Park Wastewater Treatment Facility in whose system two overflow retention facilities constructed by Aqua Store were inspected and Penn Hills where one unit was inspected. One unit at Plum Borough was a 2,000,000 gallon equalization facility serving the WWTP. The other, which was a bit larger than a 2,000,000 gallon facility, was placed on an interceptor line and allowed for diversion of flows from the interceptor line once they reached certain levels. The Penn Hills' unit was also located on an interceptor line and accepted flows again diverted from the sewer to a facility of approximately 2,000,000 gallons. The Penn Hills unit's diversion was originally based on flows but later switched to level. Both municipalities use constant speed pumps to fill.

None of these facilities had covers. The major difference between them is that the Plum Borough facilities had ladder accesses and an overhead walkway to the center of the tanks where the cleaning cannons were located, and the Penn Hills facility had two sets of steps that ran up the side of the tanks 180 degrees from themselves and the cannons were mounted on the side of the tank. In all cases, once the overflow event was over, the flows were metered back into the system. Each of the tanks was provided with an overflow with a pipe to the stream. The employees indicated that the County regulatory agency was satisfied and did not consider it a violation if the tank were to overflow during an event. The two Plum Borough tanks were actually discharged at the bottom of the tank but the discharge piping ran to the top made a goose neck and then back down to the bottom (constant static head). One of the tanks was provided with an air relief valve. The Penn Hills tank influent was through the floor of the tank. All tanks emptied from the middle of the tank and all tanks were quite clean although the operators indicated that they had not actually been cleaned for a period of time. The sides of the tank being the Aqua Store equipment were clean except for some nonobjectionable staining. Floors of the tank likewise were relatively clean attributed to the dilute waters discharged to the tank. The operators in all cases had little objection to the tanks, even those that had to climb the ladders. The ladders were not objectionable for climbing. The writer climbed both tanks and they were approximately 30 ft. to 34 ft. in height. Our tank will be closer to 36 ft. to 37 ft. The

Pennsylvania Department of Environmental Protection

230 Chestnut Street
Meadville, PA 16335-3481

JUL 1 2 2005

Northwest Regional Office

814-332-6942
Fax: 814-332-6121

Millcreek Township Supervisors
3608 West 26th Street
Erie PA 16506

Re: Act 537 Special Study Addendum
Kearsarge Pump Station Upgrade
Millcreek Township, Erie County

Dear Supervisors:

The Department hereby approves the Act 537 Special Study Addendum for the Kearsarge Pump Station. The Addendum was received by the Department on June 29, 2005, and was prepared by Consoer Townsend Envirodyne Engineers, Inc. on behalf of Millcreek Township and the Millcreek Township Sewer Authority. The Addendum was submitted to document the proposed changes to the June 2004 Special Study that was approved by the Department on September 30, 2004.

The changes outlined in the Addendum consist of an increase in storage capacity from 0.5 to 2.3 MGD. It also proposes an increase in the storage pumping capacity from 2340 gpm to 4500 gpm. There are no changes with regards to cost estimates, funding, user rates, implementation schedules or environmental consistency.

The Department will hold Millcreek Township and the Millcreek Township Sewer Authority responsible for the implementation of the proposed changes.

If you have any questions regarding this letter, please contact Eric Kicher of my staff.

Sincerely,

Ricardo F. Gilson
Regional Manager
Water Management

cc: George Riedesel
Erie County Department of Health
Gerald Allender, P.E.
William Steff
J. Hill
E. Kicher
K. Hoesch
M. Zimmerman
File – Planning/Act 537